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EFFECTIVENESS OF TRAFFIC CONGESTION MANAGEMENT ON INTERSECTION DELAY

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ABSTRACT

As traffic volume increases in the major cities in the world, city planners have to look for solutions to deal with congestion. Due to lack of enough land, adding new travel lanes to increase capacity is not an easy task. However, efficient usage of the existing infrastructure which doesn't cost much could be a proper solution to overcome congestion issue. In urban areas a possible way is optimizing traffic signals timing. Recent study by Roshandeh *et al.* (2013) has shown that by applying signal timing optimization models that consider both vehicle and pedestrian delays in the Chicago Central Business District (CBD), vehicle delays and travel times could be reduced by up to 10 percent when considering vehicle delays only.

The current study demonstrates that to achieve an optimal system, in general, a certain weight could be given to vehicle and pedestrian delays when applying signal timing optimization models. To do this, a measure capable of capturing the interaction between vehicles and pedestrians is studied: intersection delay. Intersection delay indicates the average delay suffered by all vehicles and pedestrians arriving at an intersection. The obtained results show that the optimal weights that could be given to vehicle and pedestrian delays when optimizing traffic signal timings are 78 percent and 22 percent, respectively. The important conclusion is that these values do not depend on vehicle and pedestrian volumes arriving at the intersection, since the measure of intersection delay is able to capture the interaction between them to determine the optimal priority weights. When using these weights in the optimization model, reduction on intersection delay by up to 15 percent is achieved.

Keywords: Optimization, Intersection, Shockwave, Delay, Signal

RESUMEN

A medida que el volumen de tráfico incrementa en la principales ciudades del mundo, los gestores urbanos se ven obligados a buscar soluciones para tratar la congestión. Debido a la falta de espacio en las ciudades, de alta densidad poblacional, añadir más capacidad en forma de nuevos carriles no siempre es posible. A pesar de ello, el uso eficiente de la infraestructura actual no implica un coste adicional y es una buena solución para solucionar el problema de la congestión. En áreas semaforizadas una posible solución viene por optimizar los "timings" de dichos semáforos. Estudios recientes por Roshandeh *et al.*. (2013) muestran que aplicando modelos de optimización semafórica que tienen en cuenta tanto las demoras de vehículos como de peatones en el Chicago Central Business District (CBD) es posible reducir dichas demoras en un 10%. El presente estudio demuestra que para llegar a un óptimo del sistema, en general, debe darse un peso relativo concreto a las demoras de vehículos y peatones cuando se aplican modelos de optimización semafórica. Para llegar a dichos pesos, se estudia una medida capaz de representar la interacción entre vehículos y peatones: las demoras en las intersecciones. La demora en la intersección muestra la demora media sufrida por todos los vehículos y peatones que llegan a una intersección. Los resultados obtenidos muestras que los pesos óptimos que deben darse a las demoras de vehículos y peatones cuando se utilizan modelos de optimización semafórica son del 78 por ciento y 22 por ciento respectivamente. La más importante conclusión que se deriva de este estudio es que dichos valores no dependen de los volúmenes de vehículos y peatones que llegan a la intersección. Utilizando dichos pesos en el modelo de optimización en el Chicago CBD las reducciones en las demoras llegan al 15 por ciento.

Palabras clave: Optimización, Intersección, Onda de choque, Demora, Semáforo

CHAPTER 1

INTRODUCTION

One of today's biggest challenges in urban environments is the management and mitigation of traffic congestion. The urban streets of today's big cities are operating very close to capacity and the impossibility to increment this capacity due to spatial limitations makes urban managers responsible of getting the most out of the resources they have. This implies using quantitative methodologies that can help us achieve the optimal use of the existing system and provide the best level of service for urban travelers.

The Chicago metropolitan area is well known for having traffic congestion issues. Several work and studies have been done around traffic signal timing plan optimization in order to mitigate vehicle delays in the network, applying different methodologies and obtaining different results. Part of traffic models come from kinematic wave theory and some work has been done about it. Although most models involve mitigating vehicle delay only, new innovative research by Roshandeh *et al.* (2013) developed a methodology that not only involves vehicle delay but also incorporates pedestrian delay in the objective function. This makes a lot of sense especially in areas with high density of pedestrians, which is a fact in downtown areas of most touristic and business cities in the world, Chicago not being an exception.

Obtained results show that optimized traffic signal timing plans can reduce vehicle travel time and delay. They also show that this reduction is higher when the relative importance that is given to vehicle delays is higher in front of pedestrian delays. This is captured in the performed sensitivity analysis, which associates different weights to both types of delays. Thus, these two measures (vehicle travel time and delay) do not capture the real performance of the system since it is suggesting the manager of the urban network to neglect pedestrian delay in order to obtain the highest reductions in vehicle delays, which obviously makes sense.

Observing these results also suggests that another driver must be observed to find an optimal relative weight of vehicle versus pedestrian delays in order to have the system running and using its resources effectively. In other words, the interaction between different vehicles needs to be considered as well. This paper focuses on the concept that best captures the interaction between vehicles and pedestrians to finally find out link (road segment)'s travel time and delay reductions.

However, intersection delay and its variation is another main issue which should not be forgotten when the proposed optimization methodologies are applied. Intersection delay is defined in the Highway Capacity Manual (HCM) (TRB, 2010) as "Total additional travel time experienced by vehicles as a result of optimizing traffic signals timing and interaction with other users, divided by the volume departing from the corresponding cross section of the intersection". Signals timing optimization may cause intersection delay to either go up or down.

The main objective of this thesis is to analyze intersection delay when the optimized traffic signal timing plan methodology proposed by Roshandeh *et al.* (2013) is applied. Furthermore, this study wants to prove that there should be an optimal weight that could be given to vehicle and pedestrian delays respectively in order to achieve the system's optimal performance. It seems intuitive that vehicle delay should have a higher relative weight, but, how much higher? What is the exact weight that should be given in order to have the system running effectively?

Determining that weight will help urban and traffic planners design their traffic signal timing plans and contribute to the mitigation of traffic congestion on today's cities.

This thesis is organized in five chapters as follows:

Chapter 1: Introduces the concept of intersection delay and proposes the main study objectives.

Chapter 2: Presents a literature review and state of the art of the most innovative research work related to intersection delay. Different proposed models to calculate intersection delay are studied.

Chapter 3: Describes the proposed methodology that is used for calculating intersection delay.

Chapter 4: Provides details of the applied model. A focus on data filtering and preparation is given since its treatment has been important to establish the model.

Chapter 5: Presents the application of the model and the obtained results. Partial but interesting results are also discussed to see the turning points in the research process.

Chapter 6: Summarizes the most important results and states the obtained conclusions. Further research on the topic is also proposed.

CHAPTER 2

LITERATURE REVIEW

The review of the available literature will be divided into two different sections. The first section will analyze the research work that is the starting point of this thesis. As mentioned, the aim of this study is to show how the traffic signal timing plan optimization model proposed by Roshandeh *et al*. (2013) affects intersection delay. Thus, a detailed review of that work is made in the first section. The second part of this chapter will focus on work related to the calculation of intersection delay in order to develop the best model possible. A lot of research has been done on creating models to calculate intersection delay and while the standard and vastly used model, especially outside of the academic world, is the HCM model, several other models have been developed using different mathematical tools.

2.1 Traffic Signal Timing Plan Optimization Model

Roshandeh *et al*. (2013) propose a new methodology for signal timing optimization to minimize total vehicle and pedestrian delay, by adjusting green splits in each intersection, without changing the existing cycle lengths. Two things are innovative about this methodology: first, it not only analyzes vehicle delay like the rest of the models, but also the interaction with pedestrians and other transport modes, Second, it makes the analysis and optimization in the whole system and not only in a single corridor or an isolated intersection.

Two models are studied: i) the Basic Model that only includes vehicle delay; and ii) the Enhanced Models that include also pedestrian delay. The goal is to minimize the average delay of the whole system, and for that reason an objective function is defined for each model. This objective function contains different vehicle and pedestrian delays as shown in Table 2.1:

Table 2.1: Different models in Roshandeh *et al*. (2013)

	Vehicle Delay	**Pedestrian Delay**
Basic Model	Yes (new developed model)	No
Enhanced Model 1	Yes (new developed model)	Yes (HCM method is used)
Enhanced Model 2	Yes (new developed model)	Yes (HSL method is used)

The vehicle delay equation that is used in all the models is the one proposed by Roshandeh *et al*. (2013) and that comes from the kinematic wave theory. Calculating queue lengths and wave speeds for both undersaturated and oversaturated conditions they come up with two delay equations as objective functions that combined with a constraint aiming to minimize the delay functions.

Three different situations are set for pedestrian delay. The Basic Model doesn't consider any pedestrian delay, while the first Enhanced Model uses the HCM method for calculating pedestrian delay and the second one uses the HSL (Herbert S. Levinson) method. A difference between the two is that the latter takes into account that pedestrians arriving on green time don't have to wait, while the former does not make this consideration. Also, the HSL Model considers number of pedestrians crossing the intersection while the HCM Model just calculates and average pedestrian delay taking into account green, red and yellow times.

As said, the innovation in this research is the way of finding a solution that minimizes vehicle and pedestrian delay simultaneously. The way both delays are set up together in the objective function for both enhanced models is shown in equation (2-1):

$$\text{Min} \quad w \cdot DELAYS_{VEH} + \left(1-w\right) \cdot DELAYS_{PED} \tag{2-1}$$

where

$DELAYS_{VEH}$: Vehicle delay (sec). It is calculated the same way for both Enhanced models as shown in Table 2.1;

$DELAYS_{PED}$: Pedestrian delay (sec). Calculated differently for two Enhanced models as shown in Table 2.1; and

w : Vehicle delay weight (in percent). It is the relative importance assigned to vehicle delays to determine optimal signal timing plans yielding to the lowest level of average overall delay per cycle. It varies between 10 and 100 percent, representing the two extreme cases of emphasizing vehicle delays or pedestrian delays only.

An iterative process is then applied and new green splits are obtained. New timings are then applied to a modeled traffic network through the Transportation Analysis and Simulation System (TRANSIMS) model and travel times and delays are found. The variations, positive or negative, between these travel times and delays before and after the optimization are used to tell the usefulness of the methodology. Also, a sensitivity analysis is made using different relative weights for both the vehicle delay and the pedestrian delay.

The obtained results show that the effectiveness of the model depends vastly on the area and traffic conditions where it is applied. Applied to the Chicago Business

District it shows that some areas could see delays reduction by 13 percent when applying the proposed methodology. Another important result comes from the sensitivity analysis. As commented in the introduction, it is shown that the greatest reductions in vehicle travel time and delay are obtained when the basic model is applied, which gives vehicle delay a weight of 100 percent.

The basic and enhanced models were applied to the Chicago Central Business District (CBD). The studied area was split in four parts: the core area of Chicago Loop bounded by Wacker Drive along the Chicago River, Roosevelt Road and Lakeshore Drive (Area 1); the near north of Loop bounded by the Chicago River, North Avenue and Lakeshore Drive (Area 2); the Near West Loop bounded by I-90/94, the Chicago River, North Avenue and Roosevelt Road (Area 3) and the West Loop bounded by Ashland Avenue, I90/94, North Avenue and Roosevelt Road (Area 4). Details are shown later in this report in Figure 5.1.

In Figures 2.1, -2.4, when two or more digits appear in the legend, it means that the associated curve shows results for a combination of two or more areas. For instance, curve 34 is showing the combined results for areas 3 and 4.

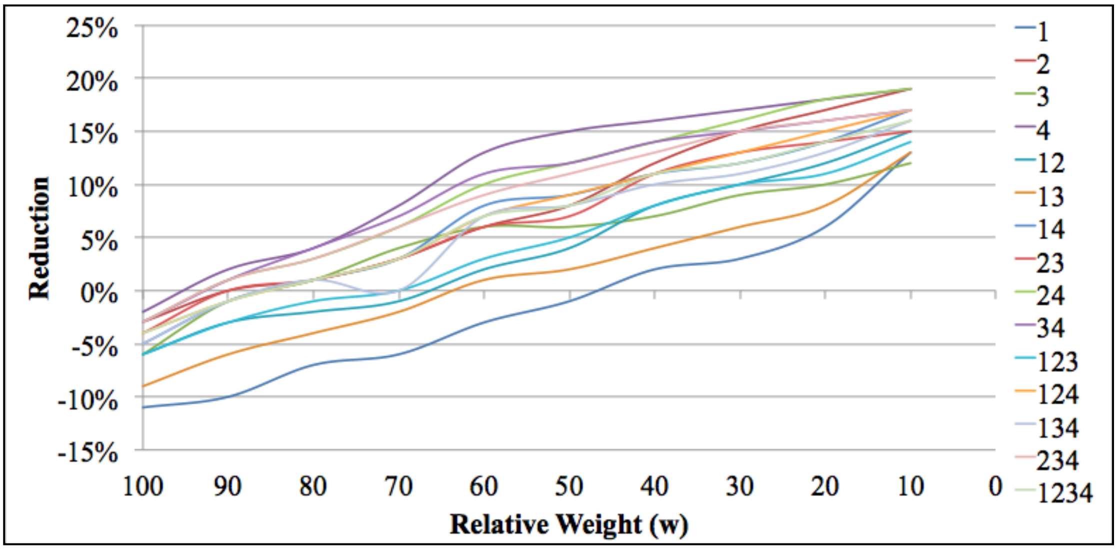

Figure 2.1*: Reductions in Travel Time Using the First Enhanced Model in Roshandeh *et al.* (2013).

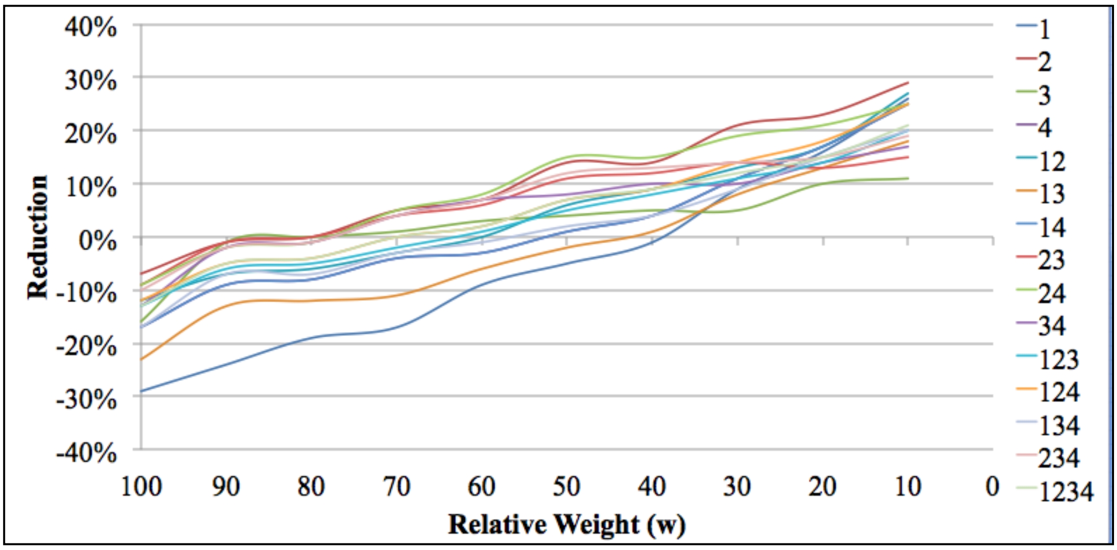

Figure 2.2: Reductions in Vehicle Delays Using the First Enhanced Model in Roshandeh *et al.* (2013).

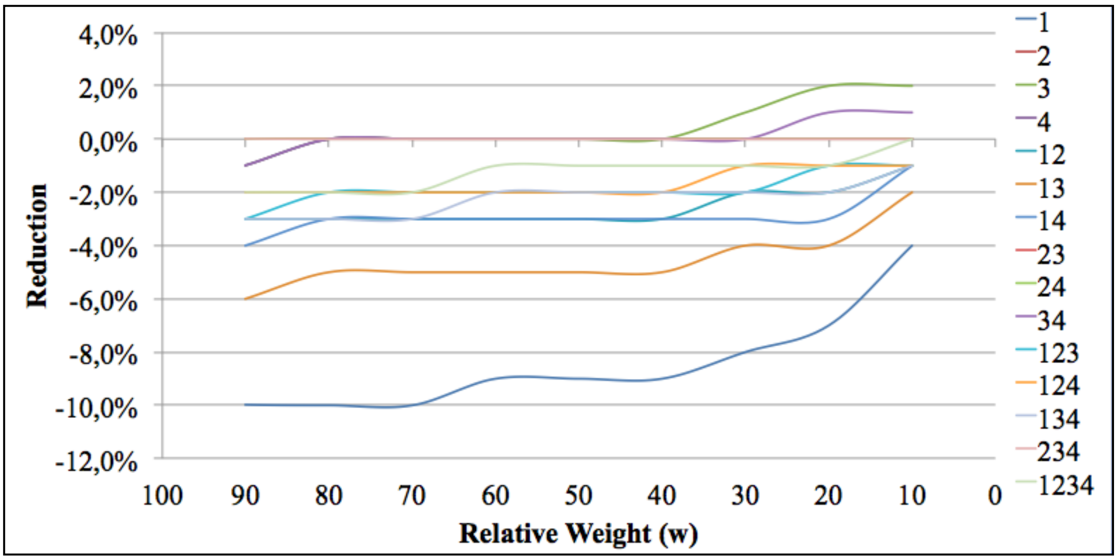

Figure 2.3: Reductions in Travel Time Using the Second Enhanced Model in Roshandeh *et al* .(2013).

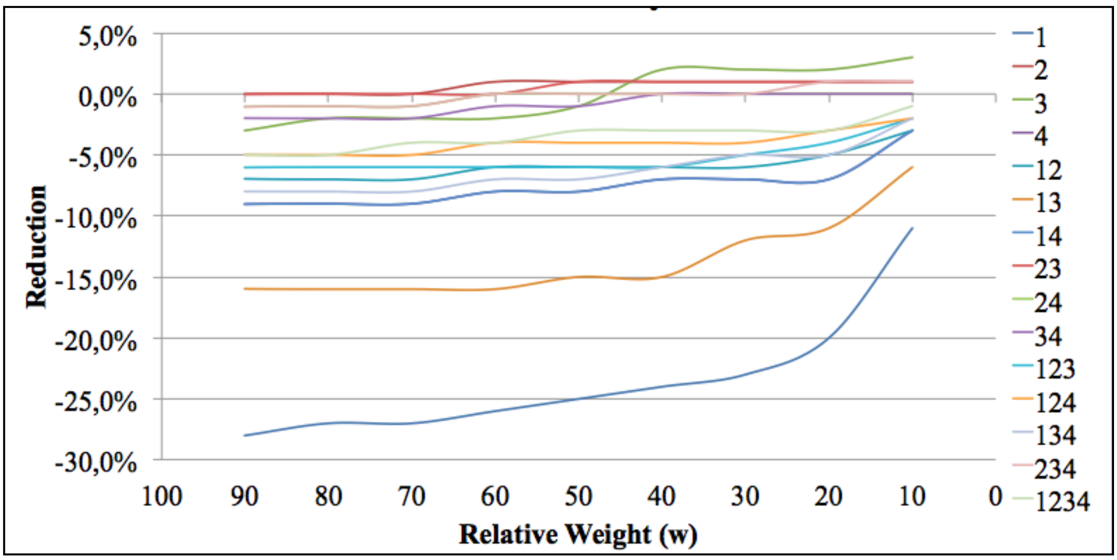

Figure 2.4: Reductions in Vehicle Delay Using the Second enhanced model in Roshandeh *et al* (2013).

As it can be observed from Figures 2.1, - 2.4 there is a clear linear relationship between the reductions on vehicle delay and travel time and the weight (*w*) given to vehicle delay in each of the areas and any the combination of them (1, 2, 3 and 4). This is especially strong in the first enhanced model which used HCM method for calculating pedestrian's delay, as seen in Figure 2.1 and 2.2. This opens the new research line that is followed in this thesis by looking at another driver, intersection delay, and tries to find an optimal relative weight for both delays. Different ways this intersection delay can be calculated are presented in the following section.

2.2 Intersection Delay Calculation Models

The current literature proposes different ways to calculate intersection delay. This section will analyze different types of proposed methods and will explain deeply each one of them.

2.2.1 Types of Proposed Methodologies

While the HCM methodology, which will be explained shortly, is the most commonly used method for calculating intersection delay, we can identify three different groups of improved methodologies in the literature:

- **HCM variations**. Most of the studied papers try to improve the HCM methodology by applying different parameters that adjust better the on-site measured intersection delay with the obtained results of the HCM methodology. These improved methods have two characteristics: first, they are usually based on countries outside the United States where HCM hypothesis might not be valid or might differ somehow; second, some of them have had results that have been incorporated in the current HCM 2010 methodology.

- **Experimental methodologies**. Several other papers deal with developing methodologies that calculate intersection delay through empirical data observation. Either by Lagrangian tracing through GPS in each vehicle or through Eulerian tracing by analyzing images taken at intersections, they are able to correlate the obtained data with intersection delay.
- **New mathematical models**. A more selected group try to calculate intersection delay through other mathematical tools. An example of this is the fuzzy logic applied in Qiao *et al*. (2002). These models might need data that is not used in other types of methodologies.

Having briefly reviewed the three main categories, the following section will analyze each one of them and study the most relevant papers.

2.2.2 HCM Variations

Koutsopoulos and Habbal (1994) studied how different ways to calculate intersection delay affect the results obtained when running a traffic assignment model. They used the HCM 1985 procedure for calculating intersection delay and studied 4 scenarios with different levels of detail on intersection geometry and data. Their first conclusion is that the HCM procedure works well, and the second one is that although level of detail reduces error significantly, computational requirements make it too prohibitive (1hr vs. 36 hr.) Thus, the simple approach is a good option when using an assignment model. However, the current computational capabilities almost 20 years later might let us use a greater level of detail without much trouble.

Fambro and Rouphail (1997) proposed an improvement on the 1994 HCM equations to calculate intersection delay, which is adopted in later versions and lead to the equations in the current HCM. They began by studying the current HCM equations and procedures and then proposed different improvements. Another term in the delay equation was added as well as some parameters inside the equations that adjust the model better on oversaturated conditions, with traffic-actuated control and dealing with upstream signal effects.

They verified these improvements with both field and simulated data and concluded that they should be added to the HCM as the results were adjusted better to reality and could deal with more conditions than with the previous equations.

Aashtiani and Iravani (1999) proposed a method to calculate intersection delay in the city of Tehran. They used the Webster equation (Webster ,1958) and simulated through the EMME/2 software (INRO ,1994) comparing the output with observed field data with good results. The main feature of their model is the low amount of input data needed and thus it makes it a good option for cities where data is not available or during a planning period.

Liu *et al*. (2007) proposed a calculating method that is pioneering in the sense that it divides the three traffic conditions (under-saturated, critical-saturated and over-saturated) into six traffic situations depending on the existence or inexistence of an initial queue at the intersection. The applied methodology consists first on determining the delay formula for each of the six cases, and then applying it to a three-phase signal controlled intersection. Two types of intersections are studied: when all approaches at the intersection are under-saturated and when all approaches are over-saturated.

The obtained output is the formula to use in each case as well as the way to choose it. However, further research is open since the method only works for completely under or over saturated intersections, something that does not match reality.

Mazloumi *et al*. (2010) proposed a function to calculate intersection delay in traffic assignment models considering drivers behavior. What is remarkable about this function is that it has modest data requirements, which makes it very useful for real life traffic assignment. Their research is based and calibrated on Iranian drivers behavior and might not be the case in other countries. As the HCM hypothesis on intersection delay cannot be assumed in Iran, they propose a new function and compare its results and its adjustment to reality with the HCM function.

This function and the HCM function are then introduced to a traffic assignment model in order to obtain traffic flows. When compared to real traffic data in the Iranian city of Mashhad, a better estimation of reality comes from the proposed function (R^2= 0.850 vs. R^2= 0.532). The main conclusion of the paper is that the special circumstances in each country affect intersection delay and implies that the function used to calculate this delay needs to be adjusted.

2.2.3 Experimental Methodologies

Wolshon and Taylor (1999) analyzed changes in intersection delay when applying a real time, time responsive signal control strategy (Lowrie ,1990) instead of optimized pre-timed intersection timing. This analysis is innovative since it questions the hypothesized effectiveness of real time signal control and compares it to regular pre-timed control strategies.

This analysis was applied in six intersections in the city of South Lyon due to optimal evaluation conditions. In order to compare both strategies a function is defined in order to calculate intersection delay. The obtained results show that using the SCATS system the delay is better distributed among the different intersection approaches, but, in some cases, this resulted in an increase of the total intersection delay, which shows an unstable effectiveness of SCATS.

Quiroga *et al*. (1999) also proposed a new methodology to calculate intersection delay. They installed GPS units in several cars and thanks to its precision were able to effectively determine deceleration, acceleration and stopped delay times. They could study the relations between them much better than other methods. They concluded that there is a linear relationship between these delays as well as different constant values for deceleration and acceleration.

Li *et al*. (2009) proposed a new method to calculate intersection delay through image processing. Their research stands out because it is the first that uses image processing in order to calculate intersection delay without having problems with overlapping vehicles and without having to be calibrated by a simulation model first.

They use an algorithm to effectively detect and track vehicles, using two camera detectors, one upstream of the intersection and the other one at the stop line. Then they calculate the delay of each vehicle, as the difference between the time a vehicle passes the stop line at the downstream line and the time it would take the vehicle to cross that distance at free speed. This individual delays and then averaged obtaining intersection delay. Obtained results compared with manually obtained data verify the effectiveness of the method, as well as showing the usefulness of the algorithm in real-time application due to its low time consumption.

Zhu *et al*. (2011) proposed a real-time road network model to be applied on vehicle navigation that considers intersection delay. The significance of this new model is not only that it includes intersection delay, something others don't consider or consider a constant, but it also re-optimizes the vehicles path every time new information is gathered and in a simpler way than other methods.

The methodology applied consisted on improve the Dijkstra algorithm by updating its optimal once new information is obtained. This model was then tested on the city of Chongqing in China obtaining good results in the optimality of the path as well as the matching with reality. Its major problem, however, is the gathering of the data since it must come from vehicles in the public transport fleet (taxis or vehicles) and thus hard to get.

2.2.4 New Mathematical Models

Qiao *et al*. (2002) proposed a methodology to calculate intersection delay through fuzzy logic. The big difference with the rest of methods for calculating intersection delay is that using fuzzy logic enables to not only include technical factors but also non-technical factors that affect delay such as weather. They set up a fuzzy model that includes as input flow ratio, green time, cycle time and weather.

After setting up the fuzzy model they ran it alongside with the Webster model and the HCM model comparing them to field data on an intersection in Hong Kong. The error of the fuzzy logic model is significantly lower than the errors for the Webster and HCM models and it is then concluded that it is a model that should be considered when calculating intersection delay.

CHAPTER 3

PROPOSED METHODOLOGY

After reviewing the different methodologies that have been developed for calculating intersection delay, this chapter will define the one that is going to be used. It will justify why it is chosen and the hypothesis that have been made in order to apply it. It also defines the data that is needed to use this methodology.

3.1 Justification of the Proposed Methodology

Focusing on the main objective of this thesis, an appropriate methodology needs to be chosen. After reviewing different types of methodologies, this work chose the HCM 2010 model for analyzing intersection's delay. This decision is made for the following reasons:

1. The HCM model is widely used in both the academia and the industry. Although it might not be valid in some countries as drivers behavior is very different, the HCM model has seemed to work well in the U.S.A. Taking into account that this study will calculate intersection delay in downtown Chicago, the HCM model seems a good choice.

2. The available data is the output of the TRANSIMS model application in Roshandeh *et al*. (2013). TRANSIMS gives results on a lot of variables, but the transcendental ones in intersection delay deal with speed, volume, maximum queue lengths and travel times. There is also a big constrain on input data. This means that a methodology cannot be chosen that takes input that differs from these variables. This also means that neither the empirical methodologies can't be used nor the new mathematical models like Qiao *et al*.

(2002), that have input variables like weather that are not results from TRANSIMS modeling.

3. The hypotheses of the HCM methodology are applicable in the Chicago network as will be shown in the following section. This makes the HCM methodology useful in this case.

3.2 HCM Methodology for Calculating Intersection Delay

The HCM 2010 defines intersection delay as the average control delay experienced by all vehicles that arrive during the analysis period. It includes any delay incurred by these vehicles that are still in queue after the analysis period ends. The methodology calculates intersection delay the following way:

$$d = d_1 + d_2 + d_3 \tag{3-1}$$

where

d : Intersection (control) delay (s/veh);

d_1 : Uniform delay (s/veh);

d_2 : Incremental delay (s/veh); and

d_3 : Initial queue delay (s/veh).

Each type of delay and the hypotheses assumed for their calculation are explained in the following sections.

3.2.1 Uniform Delay

Uniform delay is an estimate of delay assuming random uniform arrivals, stable flow and no initial queue. It also assumes one effective green time during the cycle and one saturation flow rate during this period. Saturation flow rate is the equivalent hourly rate at which previously queued vehicles can traverse and intersection approach under prevailing conditions, assuming that the green signal is available at all times and no lost times are experienced, in vehicles per hour. This means that a flow rate higher than the saturation flow rate will lead to a queue at the end of the cycle.

Uniform delay can the be calculated as:

$$d_1 = \frac{0.5C(1 - g / C)^2}{1 - [\min(1, X)g / C]} \tag{3-2}$$

where

C : Cycle length (sec). Cycle length refers to the period of time needed to complete the sequence of signal indication. Each approach in the intersection has each own cycle length;

g : Effective green time (sec). This is the available green time to perform each movement in each approach of each intersection;

$X = \frac{v}{c}$: Volume to capacity ratio. It indicates how close to saturation is each approach or lane working;

v : Traffic volume (veh/h). Number of vehicles using each lane in the approach per unit of time; and

c : Capacity (veh/h). It is the maximum sustainable flow rate at which vehicles can be expected to drive through the approach during a specified time period.

3.2.2 Incremental Delay

Incremental delay, as described in 2010 HCM, is used to estimate the extra delay due to non-uniform arrivals and temporary cycle failures (random delay) as well as delay caused by sustained periods of oversaturation (TRB, 2010). The equation assumes that there is no unmet demand that causes initial queues at the start of the analysis period. It consists of two components. The first one is associated to the fluctuations in demand during the cycle, taking into account that it can exceed capacity at some point. The second component accounts for the delay caused by a sustained oversaturation during the analysis period. It is calculated as:

$$d_2 = 900T\left[\left(X-1\right)+\sqrt{\left(X-1\right)^2+8KIX/cT}\right] \tag{3-3}$$

where

T : Duration of the analysis period (sec). It is the period of time during which we consider each variable and parameter a constant;

k : Incremental delay factor. It is a factor used to incorporate the effect of the type of signal control. For pre-timed signals HCM (2010) recommends a value of 0.5 (obtained from experimental tests); and

I : Upstream filtering/metering adjustment factor. It incorporates the effects of metering arrivals for upstream signals. For the analysis of an isolated intersection a value of 1.0 is suggested in the 2010 HCM.

3.2.3 Initial Queue Delay

The equation used to estimate incremental delay is based on the assumption that there is no initial queue present at the start of the analysis period. The initial queue delay term accounts for the additional delay caused by an existing initial queue. This queue is the result of unmet demand in the previous time period. Initial queue delay is computed as:

$$d_3 = \frac{3600}{vT}\left(t\frac{Q_b + Q_e - Q_{eo}}{2} + \frac{Q_e^2 - Q_{eo}^2}{2c} - \frac{Q_b^2}{2c}\right) \quad (3\text{-}4)$$

with $$Q_b(i) = Q_b(i-1) + T(v(i) - c) \quad (3\text{-}5)$$

$$Q_e = Q_b + t(v - c) \quad (3\text{-}6)$$

if $v > c$, then $$Q_{eo} = T\left(v - c\right) \quad (3\text{-}7)$$

$$t = T \quad (3\text{-}8)$$

if $v < c$, then $$Q_{eo} = 0.0veh \quad (3\text{-}9)$$

$$t = \frac{Q_b}{c - v} \quad (3\text{-}10)$$

where

i : Period of calculation. It takes values from 1 to 24 since we worked on 24 analysis periods of 1 hour each to compute for all day;

t : Adjusted duration of unmet demand in the analysis period (h). This is the period of time where the approach is working on dissipating a queue. If traffic volume is higher than capacity the initial queue will never disappear while it will if capacity exceeds traffic volume;

Q_b : Initial queue at the start of the analysis period (veh);

Q_e : Queue at the end of the analysis period (veh); and

Q_{eo} : Queue at the end of the analysis period when traffic volume exceeds capacity and there is no initial queue (veh).

The important difference between initial queue delay and uniform and incremental delay is that initial queue delay is computed directly for the whole approach, and not for every lane group like with d_1 and d_2. This is because initial queue delay takes as inputs different types of queues, and the distribution of these queues among the different lanes is unknown. Thus, when calculating total approach delay, while d_1 and d_2 will have to be averaged in the approach, d_3 will just be added since it will already be a delay associated to the approach and not a lane group.

CHAPTER 4

MODEL DESCRIPTIONS

This chapter will focus on defining the model used in this research that follows the HCM (2010) intersection delay calculating methodology defined in Chapter 3. The first section of this chapter will explain how the TRANSIMS simulation tool is used and how it works since it is the one providing the used data in this research. The second part will show how the output of the TRANSIMS model obtained by Roshandeh *et al*. (2013) is prepared in order to be input-ready data for our algorithm. The third part of the chapter will explain how the algorithm developed in this study works. Then, the fourth part will explain the way results are aggregated and treated. The last part of the chapter will show a simple example proving the effectiveness of the algorithm.

4.1 TRANSIMS Simulation Tool

Before defining how the data was set-up to run the model effectively it is important to understand where it comes from: TRANSIMS. TRANSIMS is an abbreviation for TRansportation ANalysis and SIMulation System, and is an integrated set of tools to conduct regional transportation system analysis based on a cellular automata microsimulator. It is capable of conducting large-scale simulation on a second-by-second basis for detailed regional multimodal transportation planning, traffic operations and evacuation planning/emergency management analyses. Its approach is based on modeling individual travelers and their multi-modal transportation based on synthetic populations and their activities. Compared to traditional traffic planning approaches, TRANSIMS requires a significant amount of data and computing resources.

Li *et al*. (2012) used TRANSIMS to calibrate and validate the Chicago model using field data on traffic volumes, speeds and travel times in the Chicago Metropolitan Area collected from over 800 continuous traffic counting stations and 1,200 urban street mid-block counting locations.

The Chicago TRANSIMS model was first applied for regional traffic assignments using the regional daily O-D travel demand matrix and the existing (i.e., the original situation in this study) traffic signal timing plans. The existing traffic signal timing plans were obtained from the Chicago Department of Transportation.

Next, Roshandeh *et al*. (2013) applied the iterative process from their model to obtain new signal timing plans by adjusting the existing green splits of all signal phases for AM peak, PM peak and rest of the day time period without changing the existing cycle lengths and signal coordination. This implies having nineteen new and different signal timing plans: the basic optimized model signal timing plan, the first enhanced optimized model signal timing plan which incorporated HCM (2010) method for computing pedestrian's delay (9 different signal timing plans associated to 9 weighting factors ranging from 90 to 10 percent) and the second enhanced optimized model signal timing plan which applied HSL method for pedestiran's delay computation (9 different signals timing plans). These new signal timing plans were then used as input set of data in TRANSIMS to be able to obtain traffic volume on each link (road segment).

TRANSIMS was then ran twenty different times, once for each new traffic signal timing plan, and one time using original signal timing plans. Basically, signal timing plan is was the only input value varying among different scenarios. Results on a lot of variables including travel times, traffic volumes, maximum queues and speeds were obtained. This study uses the traffic volume results given by TRANSIMS as well as the twenty different signal timing plans (one original and nineteen optimized) obtained in Roshandeh *et al*. (2013) in order to calculate intersection delay. The idea is to compare how intersection delay changes when applying the different optimized plans.

4.2 Hypotheses and Data Preparation

MATLAB is used to conduct this part. In order to understand how the algorithm is coded in MATLAB it is important to know how the data is set up. The delay in each intersection is a weighted average delay from all the approaches in the intersection. At the same time, the delay associated to each approach is another weighted average delay from each of the three lane groups (through lanes, right-turn lanes and left-turn lanes). Thus, the proposed algorithm is applied to each lane group of each approach on the intersection and then aggregated, as it will be shown later.

To do that, for each iteration to calculate delay the code picks up information from two Microsoft Excel files. The first one contains the time-static information of each approach and the second one contains associated volume. Information on traffic volume for each hour of the day is available, so this procedure is repeated 24 times for each intersection. There are 4 major situations: the original situation, the optimized situation with the basic model, the optimized situation with the first enhanced model and the optimized situation with the second enhanced model. Also, a sensitivity analysis is made in the enhanced models, so in each one of them the code has been run for several times.

	A	B	C	D	E	F	G
1	14826	15914	0	0	0	0	0
2	14828	15918	0	0	0	0	0
3	14829	15848	0	0	0	0	0
4	14831	15928	0	0	0	0	0
5	14834	15859	0	0	0	0	0
6	14844	15876	0	0	0	0	0
7	14846	15876	41105	15878	15879	0	0
8	14847	15880	0	0	0	0	0
9	14850	80693	15885	15886	15887	0	0
10	14851	41274	80694	15888	15889	0	0
11	14852	15888	41109	41228	15890	0	0
12	14856	41117	15898	15899	15900	0	0
13	14857	41299	80677	80690	15901	0	0
14	14861	15859	15908	15909	15910	0	0
15	14862	15860	15908	15911	0	0	0
16	14863	15911	16026	41050	15915	0	0
17	14864	15843	15915	15916	0	0	0
18	14865	15916	41052	15919	0	0	0
19	14866	15919	15920	15921	41071	0	0
20	14867	15920	41069	15922	15923	0	0

Figure 4.1*:* Snapshot of the first tab at the time-static worksheet. First column is node number and other columns are names of links arriving at the node.

The set-up of the data is illustrated in figures 4.1 and 4.2. Figure 4.1 shows the first 20 rows (out of 934, one for each node) of the first tab of the time-static worksheet for the original situation. In this worksheet, the first column corresponds to the node (i.e., intersection ID), while the rest of the columns correspond to information about each link (i.e., road segment) arriving at the node. In other words, figure 4.1 shows the first tab of the worksheet, which is the name (number or ID) of each link arriving at the node. When there is a zero it means that no more links are related to that node. For example, at node 14862 (row 15) the arriving links are 15860, 15908 and 15911. Since the maximum number of links arriving at an intersection is 6, the matrix has 7 columns (1 for nodes and 6 for links).

Other tabs are information related to each link. Thus, the basic structure of the 934x7 matrix is maintained for all of them, but obviously information varies. Apart from each links name arriving at the node, as shown in figure 4.1, the eight other tabs contain: cycle length, capacity, number of through lanes, number of left lanes, number of right lanes, green through times, green left times and green right times. Obviously, all cells in the rest of the tabs will have a value of 0 where the link tab has a value of 0 since the non-existance of a link implies no information about it. The other way implication is not true. It is possible that a link doesn't have all type of lanes, so an extra zero may appear in the number of lanes or green times if that link doesn't contain lanes for that movement. As said, this information is time-static, since it doesn't vary through the 24-hour day. However, part of this data will vary when switching between the four different situations (original, basic optimization, first enhanced optimization and second enhanced optimization). While the links arriving at the intersection, capacity of links and number of through, left and right lanes will be maintained, cycle lengths and green times will change.

Figure 4.2 shows, as an example, the first 20 rows of traffic volume information for the Original situation at the first hour of the day. Same as the time-

	A	B	C	D	E	F	G
1	14826	105	0	0	0	0	0
2	14828	23	0	0	0	0	0
3	14829	19	0	0	0	0	0
4	14831	17	0	0	0	0	0
5	14834	34	0	0	0	0	0
6	14844	125	0	0	0	0	0
7	14846	138	23	93	42	0	0
8	14847	32	0	0	0	0	0
9	14850	71	40	35	43	0	0
10	14851	37	60	124	48	0	0
11	14852	65	95	86	51	0	0
12	14856	16	14	27	18	0	0
13	14857	37	18	13	27	0	0
14	14861	26	152	235	35	0	0
15	14862	117	267	155	0	0	0
16	14863	63	140	31	84	0	0
17	14864	58	63	58	0	0	0
18	14865	55	19	53	0	0	0
19	14866	47	39	12	19	0	0
20	14867	41	36	34	27	0	0

Figure 4.2: Snapshot of the First Tab (Hour 00-01) at the Traffic Volume (Veh/H) Worksheet. First Column is Node Number and Other Columns are Volumes at Links Arriving at the Node

static information worksheet, the first column is node (i.e., intersection ID), and the rest of the columns are volume at each link. This worksheet has 24 tabs, since we obtain volume for each hour of the day. For example, for link 14862, the traffic volumes on that link arriving at the node are 117, 267 and 155 vehicles for links 15860, 15908 and 15911 respectively. As it can be seen, all matrices are consistent with each other, and information of each link is in the same position in all matrices, which makes it easy to reference when calling from the MATLAB script.

As it can be seen, the set-up of the data is very robust and makes the programming of the MATLAB script very easy. The way the data is set up allows to reference each value very easily and is the key element to simplify the code and to speed up computations.

4.2.1 Time-Static Information

Having seen the variables and parameters used by the algorithm, this section will describe each one of them. Time-static information is the information about the node that only changes, partially, between each optimization, and not during the day. It contains several 934-row matrices that contain the following information:

a) Links

Each node has associated the identification number of each link that arrives to the node.

b) Capacity

Each links capacity. It is assumed that each lane in each approach has the same capacity, since the total number of lanes divides the total capacity of the approach. Approach capacity is obtained from the Chicago GIS Network available at the Illinois

Institute of Technology. When determining the number of through, left and right lanes, the capacity obtained from the GIS Network might be modified as it will be seen in the next section.

c) Number of Through, Left and Right Lanes

Number of lanes of each link for each movement. Although some lanes might be shared between different movements, this model will suppose that each lane can only either go through, left or right. If there is just one right/left lane and it is shared, priority is given to the right/left movement. When there is more than one right or left movement lane and one of them is shared with the through movement, one of the shared lanes is considered to be a through lane and the other one a right lane. This assumption makes sense since when several lanes are available for a right/left movement, usually the majority of the traffic is on the non-shared lanes. However, this simplification is giving too much relevance to the right or left turn lanes, so the traffic volume and capacity on each lane has to be modified.

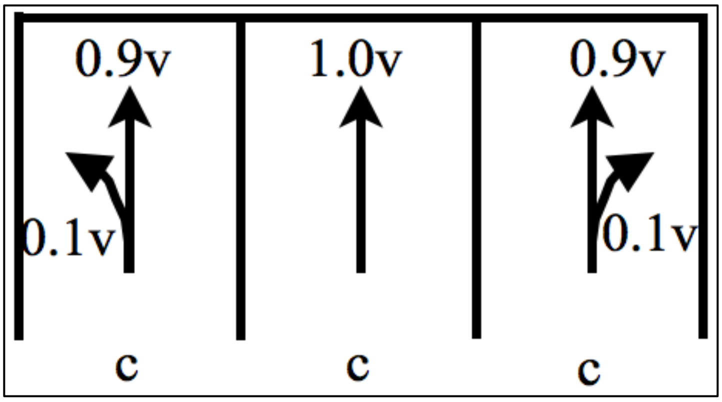

Figure 4.3. Real Lane Distribution Situation. v and c are Volumes and Capacity in that Lane

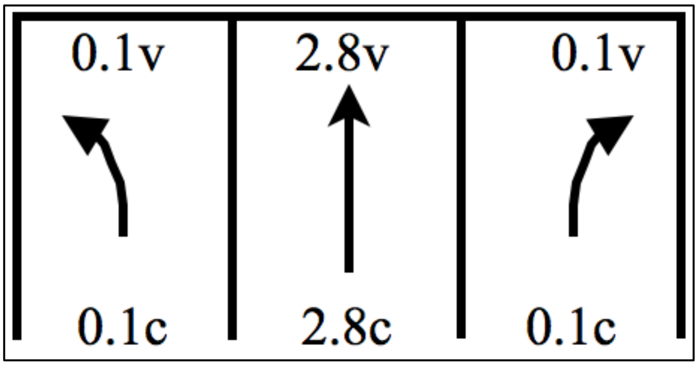

Figure 4.4. Simplified Lane Distribution Situation. v and c are Volumes and Capacity in that Lane

Figures 4.3 and 4.4 show an example of how this simplification is made. As Figure 4.3 shows, the real number of through lanes is 3, but two of them are shared with left and right turns. An assumption relating volume distribution in shared lanes is made by saying that in those lanes 90 percent of the volume goes through while 10 percent makes the left or right turn. If *v* and *c* are the volumes and capacity of each lane, Figure 4.3 shows volumes going in each direction for each lane as well as each lanes capacity. Since the simplification made in this study is that each lane only has one type of movement, volumes and capacities have to be modified. Figure 4.4 shows how volumes and capacities change in order to accommodate the new simplified situation. When considering the outside lanes only turning lanes, the through traffic of those lanes has to be transferred to the available through lanes (middle lanes in this example). However, to maintain the volume to capacity ratio (*X*), capacities have to be modified in the same proportion.

The reason why this procedure is chosen is that it maintains the results that we would obtain using the real situation but it is much easier to compute. The MATLAB code was much simpler to program if each lane had just one function, and modifying volumes and capacities was a simple task. The key that enables us to make this simplification is that all the traffic volume in each link are considered to be evenly distributed on all lanes.

d) Green Times For Through, Left and Right Lanes

Green times for each lane group. The original data from Roshandeh *et al*. (2013) contained several different timings for each node depending on the hour of the day. As a simplification, only one timing is used for each node. Also, green times were split between protected and unprotected movements. This model assumes that protected and unprotected movements are equal and their green times can be added. Since each timing has different phases and a group lane might have more than one green time in each cycle, these times will be added. The 2010 HCM recommends a possible improvement to obtain a more precise calculation by splitting green times in different phases and calculating the queue between them (TRB, 2010). Given the big amount of operations the simplified version is applied in the model.

Figures 4.5, to 4.7 show an example of how this simplification is performed. As shown in Figure 4.5, an imaginary node with reference 1000 has two timings (27006 and 27007) depending on the hour of the day that differ on the phases

NODE	TIMING	PHASE	IN_LINK	OUT_LINK	NOTES
1000	27006	1	15302	15301	Protected Left
1000	27006	2	15302	15301	Unprotected Left
1000	27006	2	15302	15084	Protected Thru
1000	27006	2	15302	15074	Protected Right
1000	27006	3	15301	15084	Protected Left
1000	27006	4	15301	15084	Unprotected Left
1000	27007	1	15302	15301	Protected Left
1000	27007	1	15302	15301	Unprotected Left
1000	27007	2	15302	15084	Protected Thru
1000	27007	3	15302	15074	Protected Right
1000	27007	3	15301	15084	Protected Left
1000	27007	4	15301	15084	Unprotected Left

Figure 4.5. Timing and Phases Associated with Each Movement in a Node

associated to each movement. On the other hand, Figure 4.6 shows the length in seconds of each phase on that node.

The simplification that is used considers that for each node there is just one timing, which corresponds to the first of them (in this example, timing 27006). Also, it doesn't differ between protected and unprotected turns, so in this case, green through time is the length of phase 2, green left time is the sum of phases 1, 2, 3 and 4 and green right time is the length of phase 2. Results are shown in Figure 4.7.

	PHASE			
NODE	1	2	3	4
1000	8	22	15	22

Figure 4.6. Length of Each Phase (sec) in a Node

NODE	Thru	Left	Right
1000	22	67	22

Figure 4.7. Green Times for Each Movement (sec) in a Node

4.2.2 Volume Data

This worksheet contains information about the traffic volume. It contains 24 tabs, one for each hour of the day. They are also 934x7 matrices and contain traffic volume for each link and each hour of the day.

4.2.3 Constant Parameters

The calculation of incremental delay (d_2) and initial queue delay (d_3) imply defining three constant parameters:

T : Duration of the analysis. Since volume data is for each hour of the day, this value is considered 1 hour.

k : Incremental delay factor. HCM (2010) suggests a constant value of 0.5 for pre-timed signalization like the studied case.

I : Upstream filtering/metering parameter. HCM (2010) suggests a value of 1.0 for isolated intersections. This means that an intersection is not affected by each contiguous intersections and this can be assumed here as a simplification.

4.3 Model Algorithm

The chosen methodology was coded in a MATLAB script in order to compute calculations due to the big amount of input data. Input data was stored in Microsoft Excel files and was called directly from the script to improve storing. Figure 4.8 shows the flowchart of the algorithm.

The algorithm has three loops. The first loop computes for each link in a node, the second loop computes for each node and the third loop computes for each hour of the day. There is also another level of calculation inside the first loop, where the algorithm computes delay for each group lane. Since the number of lane groups is three, a small finite number, no loop was needed. Inside each lane group there is a conditional instruction that allows the code to reduce the number of operations since it only calculates delay when the number of lanes in a lane group is greater than zero.

Once intersection delay is computed and stored in a worksheet, the averaged weighted final results for each area are computed through an Excel sheet.

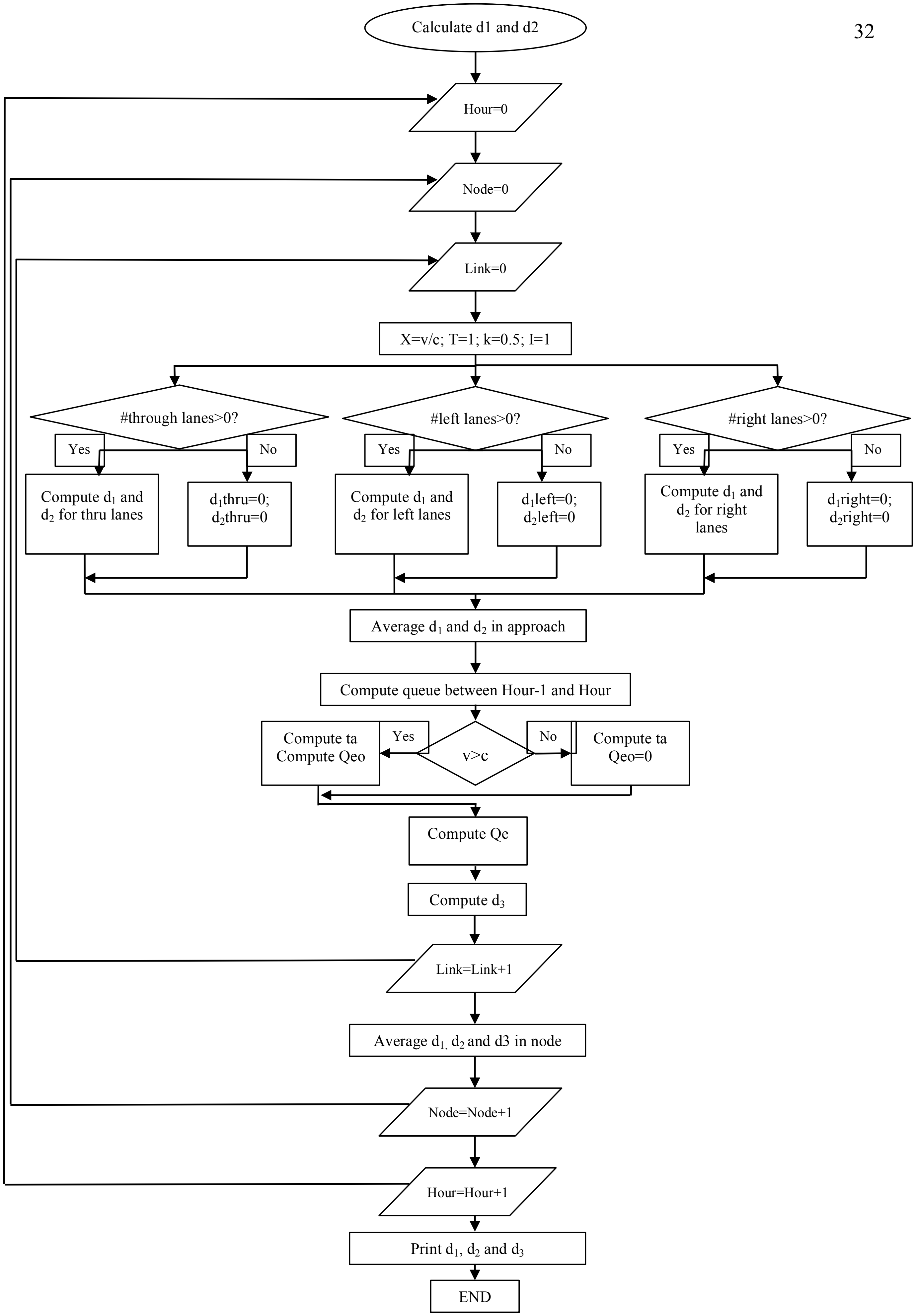

Figure 4.8. Flowchart of the algorithm used for computing intersection delay

4.4 Aggregation of Results

Once the previous algorithm is applied to each lane group for each approach in an intersection and for each hour of the day, a delay will be obtained that should be aggregated in order to obtain total intersection delay reduction. This comprises of five steps:

Approach delay. since one of the assumptions is that traffic volume is distributed evenly in each lane, approach delay will just be the mean of each lane group delay, weighted by number of lanes:

$$d_{12,thru,i} = (d_{1,thru,i} + d_{2,thru,i}) \tag{4-1}$$

$$d_{12,left,i} = (d_{1,left,i} + d_{2,left,i}) \tag{4-2}$$

$$d_{12,right,i} = (d_{1,right,i} + d_{2,right,i}) \tag{4-3}$$

$$d_{approach,i} = \frac{d_{12,thru,i} \cdot N_{thru}^{lanes} + d_{12,left,i} \cdot N_{left}^{lanes} + d_{12,right,i} \cdot N_{right}^{lanes}}{N_{thru}^{lanes} + N_{left}^{lanes} + N_{right}^{lanes}} + d_{3,i} \tag{4-4}$$

where

$d_{approach,i}$: Control delay for each approach in the intersection (sec) for the i period of time;

$d_{1,thru,i}, d_{1,left,i}, d_{1,right,i}$: Uniform delay for through, left and right lanes at each approach in the intersection (sec) for the i period of time;

$d_{2,thru,i}, d_{2,left,i}, d_{2,right,i}$: Incremental delay for through, left and right lanes at each approach in the intersection (sec) for the *i* period of time;

$d_{12,thru,i}, d_{12,left,i}, d_{12,right,i}$: Sum of Uniform and Incremental delay for through, left and right lanes at each approach in the intersection (sec) for the *i* period of time;

$d_{3,i}$: Initial queue delay for the approach in the intersection (sec) for the *i* period of time. As stated in Chapter 3, initial queue delay is calculated for each approach directly and not for each lane group; and

$N_{thru}^{lanes}, N_{left}^{lanes}, N_{right}^{lanes}$: Number of through, left and right lanes at each approach in the intersection.

Intersection delay. If an intersection has k approaches, each with different traffic volume v_k, the total intersection delay is:

$$d_{\mathrm{intersection,i}} = \frac{\sum_k d_{k,i} \cdot v_{k,i}}{\sum_k v_{k,i}} \tag{4-5}$$

where

$d_{\mathrm{intersection,i}}$: Control delay for the intersection (sec) for the *i* period of time;

k : Number of approaches at the intersection;

$d_{k,i}$: Control delay for the *k* approach in the intersection (sec) for the *i* period of time; and

$v_{k,i}$: Traffic volume at the *k* approach in the intersection (veh/h) for the *i* period of time.

Delay reduction. Once intersection delay for each node is obtained, the percent decrease between the Original and Optimized situations can be calculated. A negative number means a reduction while a positive number means an increase:

$$\%RED_i = \frac{d_{\text{intersection},i,OPT} - d_{\text{intersection},i,BASIC}}{d_{\text{intersection},i,BASIC}} \cdot 100 \qquad (4\text{-}6)$$

where

$\%RED_i$: Reduction on intersection delay (percent) for the *i* period of time and for the studied model;

$d_{\text{intersection},i,OPT}$: Intersection delay (sec) for the *i* period of time and the optimized solution; and

$d_{\text{intersection},i,BASIC}$: Intersection delay (sec) for the *i* period of time and the basic solution.

Average daily delay reduction. Once delay reduction for each node and each hour of the day is calculated an average daily value needs to be computed. Since delays are units of time per vehicle, it makes sense to average delays weighting by number of cars. That is, weighting by total traffic volume arriving at the intersection at each hour of the day:

$$\%RED = \sum_{i=1}^{24} \frac{\%RED_i \cdot V_i}{V_{TOT}} \tag{4-7}$$

$$V_i = \sum_k v_{k,i} \tag{4-8}$$

$$V_{TOT} = \sum_{i=1}^{24} V_i \tag{4-9}$$

where

$\%RED$: Average daily delay reduction (percent);

V_i : Sum of all volumes arriving at the intersection in the *i* period of time (veh); and

V_{TOT} : Sum of all volumes arriving at the intersection in a 24-hour period (veh).

Area average daily delay reduction. Once average daily delay reduction for each node is obtained, the average for each of the studied areas (which will be defined in Chapter 5) needs to be found. Those areas are just an aggrupation of nodes depending on their situation in the Central Business District (CBD).

$$\%RED_{AREA} = \frac{\sum_{j=1}^{n} \%RED_{AREA,j}}{n_{AREA}} \tag{4-10}$$

where

$\%RED_{AREA}$: Area average daily delay reduction (percent);

$AREA$: Name of study area. For example, if area 1 is considered, then $AREA = 1$. As it will be shown in Chapter 5, it can take the values of 1, 2, 3 or 4; and

n_{AREA} : Number of nodes in the study area

Combined area average daily delay reduction. In order to obtain average results for a combination of the studied areas, an average delay is calculated the following way:

$$\%RED_{COMB} = \frac{\sum_{AREA} n_{AREA} \cdot \%RED_{AREA}}{\sum_{AREA} n_{AREA}} \tag{4-11}$$

where

$COMB$: Combination of areas. For example, if the combined average daily delay reduction for areas 1 and 3 are needed, $COMB$ is 13.

4.5 Test Example

This section will test the model with a simple example. It will calculate the delay reduction for an imaginary node numbered as 1000 for one hour during the day. Node 1000 has three links approaching called 100, 200 and 300, so the associated row in the node-link matrix (equivalent to Figure 4.1) is shown in Figure 4.9:

NODE	LINK					
1000	100	200	300	0	0	0

Figure 4.9. Link Number for Node 1000

As explained earlier in this chapter, other tabs of the time-static data worksheet (node-link matrix is the first tab) will have information about each link in the same position as the link is in the node-link matrix. So for example if links 100, 200 and 300 have capacities of 1000, 1200 and 900 veh/h, respectively, the correspondent row for node 1000 in node-capacity matrix would be the one showed in Figure 4.10:

NODE	CAPACITY					
1000	1000	1200	900	0	0	0

Figure 4.10. Capacity (veh/h) for Node 1000

Following the same procedure and situating the data from each link in the same position as the node-link matrix, the rest of the time-static variables are shown in Figures 4.11 to 4.17:

NODE	CYCLE LENGTH					
1000	70	70	70	0	0	0

Figure 4.11. Cycle Length (sec) for Node 1000

NODE	LANETHRU					
1000	3	2	3	0	0	0

Figure 4.12. Number of Through Lanes for Node 1000

NODE	LANELEFT					
1000	1	2	1	0	0	0

Figure 4.13. Number of Left Lanes for Node 1000

NODE	LANERIGHT					
1000	2	1	1	0	0	0

Figure 4.14. Number of Right Lanes for Node 1000

NODE	GREENTHRU					
1000	23	34	32	0	0	0

Figure 4.15. Green Through Time (sec) for Node 1000

NODE	GREENLEFT					
1000	5	4	6	0	0	0

Figure 4.16. Green Left Time (sec) for Node 1000

NODE	GREENRIGHT					
1000	3	4	3	0	0	0

Figure 4.17. Green Right Time (sec) for Node 1000

Figure 4.18 shows traffic volume data. Same as the time-static data, the information for each link is in the same position as the node-link matrix. This example only is computing for one hour of the day, but the actual MATLAB code picked up data from 24 different worksheets, the 24 rows corresponding to each hour of the day for Node 1000.

NODE	VOLUME					
1000	23	27	18	0	0	0

Figure 4.18. Volume (veh/h) for Node 1000

Once obtained and defined both the time-static information and the traffic volume data, intersection delay can be calculated as described in Chapter 3.

a) Uniform delay

Looking at equation (3-2), the only variable left needed to calculate uniform is the volume to capacity ratio X. Figure 4.19 shows the value of X. Since we are assuming that traffic volume is distributed evenly through all lanes, the value of X will be the same for all of them as well as for the whole approach.

NODE	X					
1000	0.023	0.023	0.020	0	0	0

Figure 4.19. Value of X for Node 1000

We are now able to compute uniform delay for each lane group. Results are shown in Figures 4.20 to 4.22:

NODE	D1THRU					
1000	15.8987	9.3594	10.4095	0	0	0

Figure 4.20. Uniform Delay (sec) for Through Lanes for Node 1000

NODE	D1LEFT					
1000	30.2282	31.1543	29.3074	0	0	0

Figure 4.21. Uniform Delay (sec) for Left Lanes for Node 1000

NODE	D1RIGHT					
1000	32.0959	31.1543	32.0918	0	0	0

Figure 4.22. Uniform Delay (sec) for Right Lanes for Node 1000

b) Incremental delay

Taking a look at equation (3-3) it can be seen that incremental delay depends not only on the value of X but also on three constant parameters. As defined in section 4.2.3, these values are:

$$T = 1.0 hour \qquad k = 0.5 \qquad I = 1.0$$

Having defined all variables, incremental delay for each one of the lane groups can be calculated. Results are shown in Figures 4.23 to 4.25:

NODE	D2THRU					
1000	0.6051	0.5917	0.5246	0	0	0

Figure 4.23. Incremental Delay (sec) for Through Lanes for Node 1000

NODE	D2LEFT					
1000	0.6051	0.5917	0.5246	0	0	0

Figure 4.24. Incremental Delay (sec) for Left Lanes for Node 1000

NODE	D2RIGHT					
1000	0.6051	0.5917	0.5246	0	0	0

Figure 4.25. -Incremental Delay (sec) for Right Lanes for Node 1000

c) Initial queue delay

As equation (3-4) indicates, initial queue delay depends on various types of queues defined by equations (3-5) to (3-10). These queues depend on variables that have already been defined in the example except for the initial queue Q_b. When the analysis period is at its beginning stages, the initial queue gets a value of 0, but to make a more generalized example, it can be assumed that there is an initial queue. The values of these queues are shown in Figures 4.26 to 4.29:

NODE	Qb					
1000	50	47	76	0	0	0

Figure 4.26. Initial Queue (veh) for Node 1000

NODE	Qeo					
1000	0	0	0	0	0	0

Figure 4.27. Queue at the end of the analysis period for v>c and zero initial queue (veh) for Node 1000

NODE	t					
1000	0.0512	0.0401	0.0862	0	0	0

Figure 4.28. Adjusted duration of unmet demand in the analysis period (sec) for Node 1000

NODE	Qe					
1000	0	0	0	0	0	0

Figure 4.29. Queue at the end of the analysis period (veh) for Node 1000

Once getting the initial queue Q_b initial queue delay can be calculated. Following section 3.2.3, following results are obtained:

NODE	D3					
1000	4.6059	2.8248	13.0975	0	0	0

Figure 4.30. Initial Queue Delay (sec) for Node 1000

d) Approach delay

Having all the components needed to calculate control delay, control delay for each one of the approaches can be calculated. Following equations (4-1) to (4-4), the following results can be obtained.

NODE	DAPPROACH					
1000	28.8971	25.8529	32.1476	0	0	0

Figure 4.31. Approach Delay (sec) for Node 1000

e) Intersection delay

Once delay for each one of the three approaches arriving at Node 1000 in this example are calculated, the control delay in the intersection will be computed by following equation (4-5) Results are shown in Figure 4.32.

NODE	DINTERSECTION
1000	28.5488

Figure 4.32. Intersection Delay (sec) for Node 1000

f) Delay reduction

Since this example only had information about one situation (and not for both the Original situation and an Optimized situation) delay reduction cannot be calculated. But in order to make a more detailed example, let's imagine that the earlier situation was the original case (i.e., without signals timing optimization). Using the same methodology for an imaginary optimized situation, which would have different data on volumes and green times, following result as shown in Figure 4.33 is obtained:

NODE	DINTERSECTION
1000	25.4456

Figure 4.33. Intersection Delay (sec) for Node 1000 on an imaginary Optimized situation

Then, having calculated delay for both the Original and the Optimized situations, delay reduction can be calculated following equation (4-6) which shows the result in Figure 4.34.

NODE	DREDUCTION
1000	-11%

Figure 4.34. Reduction on Intersection Delay (%)for Node 1000

The result shown in Figure 4.34 is the ultimate result the one is looking for in each intersection. This example has shown the calculation of delay reduction for one node and one hour of the day. If the same procedure is performed for the 24 hours of the day (which would mean changing 24 times the volume information) the average daily delay reduction will be obtained by following equations (4-7) to (4-9).

When calculating this average daily delay reduction for all nodes in the same study area, and by following equation (4-10) the area average daily delay reduction can be computed.

If it is desired to analyze average daily delay reduction for a combination of different areas, and then obtain the combined area average daily delay reduction, equation (4-11) would be used.

CHAPTER 5

MODEL APPLICATION

Taking into account that a network of 934 nodes (i.e., intersections) is analyzed and a sensibility analysis is made for the two enhanced optimization models,varying the relative weight of vehicle to pedestrian delay between 10 percent and 100 percent, it means that around 0.5 million times of calculation is conducted for intersection delays are via applying the MATLAB code. Calculation time was around 2 hours with a 2.2 GHz processor and 2MB of RAM memory. This chapter will describe the procedure used to apply the code and analyze the obtained results.

5.1 Application Procedure

The Chicago Business District (CBD) was divided in four parts as shown in Figure 5.1. The different computations of intersection delay were applied to each node and

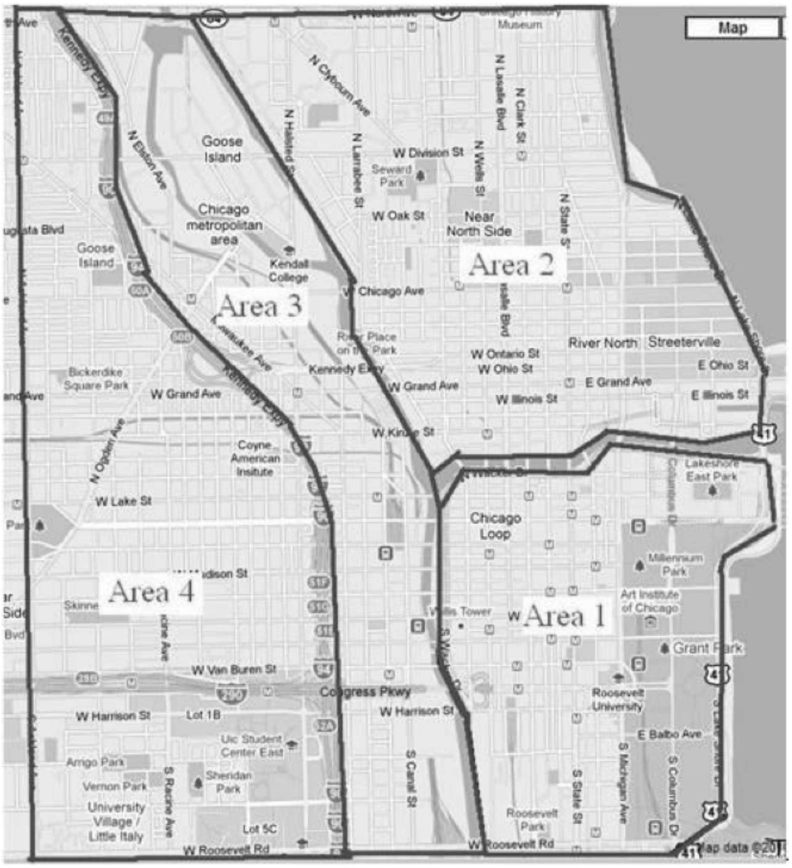

Figure 5.1. Division of the CBD in the four areas of study

then results were averaged grouping by the different areas.

The model was applied to 4 different sets of data: the Original situation, the Basic Optimized situation, the first Enhanced Optimized situation and the second Enhanced Optimized situation. The final results are the comparison (percent reduction or increase) of intersection delay between the Original situation and the three Optimized situations. Since the two enhanced optimized situations imply giving a relative weight to vehicle delay and pedestrian delay, a sensitivity analysis was made varying this relative weight (w) between 10 percent and 90 percent.

5.2 Analysis of Results

The obtained results for the different situations are shown in Table 5.1. Negative values indicate reductions while positive values indicate increases in intersection delay.

5.2.1 Basic Model

When applying the Basic model different results are obtained depending on the observed area. In some areas intersection delay goes up, in some it stays the same and in others it is reduced. Table 5.1 shows the obtained results for each of the different situations. The greatest reduction is obtained in areas 2 and 4 with reductions of -4 percent and -3 percent, respectively. On the other hand, areas 1 and 3 give the greatest increase with 7 percent and 4 percent increase in intersection delay.

Table 5.1. Intersection Delay Variation over a 24-Hour Period after Signal Timing Optimization using the Basic and two Enhanced Models

Area	Relative Weights of Vehicle Delays Ranging from 100 to 10 Percent (w)																		
	100	90	80	70	60	50	40	30	20	10	90	80	70	60	50	40	30	20	10
	Basic	First Enhanced Model (Using HCM)									Second Enhanced Model (Using HSL)								
	Variations in Intersection Delay (percent)																		
1	7	15	-4	0	5	7	8	10	11	12	13	14	15	15	15	15	15	15	15
2	-4	7	-14	-10	-8	-7	-5	-2	-1	21	5	7	5	7	6	7	7	7	5
3	4	10	-10	-6	-2	0	2	6	7	6	11	11	10	10	10	10	10	11	10
4	-3	7	-7	-5	-5	-3	-1	4	6	3	6	6	6	8	7	6	7	8	6
12	0	9	-11	-7	-4	-3	-1	1	3	19	7	9	8	9	8	9	9	9	8
13	6	13	-6	-2	2	5	6	9	10	10	12	13	13	13	13	13	13	13	13
14	0	10	-6	-4	-1	0	2	6	8	6	8	9	9	10	9	9	10	10	9
23	-2	8	-13	-10	-7	-6	-4	-1	1	18	6	8	6	7	7	7	7	7	6
24	-3	7	-11	-8	-7	-5	-3	1	2	13	5	7	6	7	6	6	7	7	5
34	-1	8	-8	-6	-4	-2	0	4	6	3	7	7	7	8	8	7	8	8	7
123	0	10	-11	-7	-4	-2	-1	2	3	17	8	9	8	9	9	9	9	9	8
124	-1	8	-9	-7	-4	-3	-1	2	4	13	7	8	7	9	8	8	8	8	7
134	1	10	-7	-4	-2	0	2	6	8	6	8	9	9	10	10	9	10	10	9
234	-2	7	-11	-8	-6	-5	-3	1	3	12	6	7	6	8	7	7	7	8	6
1234	-1	9	-9	-6	-4	-3	-1	3	4	12	7	8	7	9	8	8	9	9	7

A possible explanation of this phenomenon is the different levels of interaction between vehicles and pedestrians. Areas 1 and 3 are Chicago Loop and Near West Loop, respectively and they are the areas with the most pedestrian and vehicle interaction. Since the Basic model doesn't take into account reducing pedestrian delay, and pedestrian presence in these areas is very important, the overall intersection delay goes up. These results also show how well the concept of intersection delay is able to show the interaction of vehicles and pedestrians, since both are correlated when by the green and red times in each of the approaches. When applying the Basic model, which focuses only on vehicle delays, greater green times are given to vehicles and that might not be optimal when pedestrian presence is high.

On the other hand, when applying the Basic model on areas where vehicle presence is very high, good results are obtained. This is the case for Areas 2 and 4, West Loop and Near North Side, respectively. These areas achieve good results because the Basic model focuses only on vehicle delay and Areas 2 and 4 are mainly used by vehicles.

The results on the Basic model also show that by using the Enhanced Models greater reductions might be achieved by giving pedestrian delay a higher relative weight. The areas where vehicle presence is predominant will tend to have the greatest reductions until a certain point where pedestrian delay has a lot of importance. This is illustrated in Figure 5.2

When applying the Basic model for signal timing optimization to a combination of two areas reductions are obtained when combining areas 2 and 4 with a reduction of -3 percent, combining areas 2 and 3 obtaining a reduction of -2 percent and combining areas 3 and 4 obtaining a reduction of -1 percent. There is no change in intersection delay when combining areas 1 and 2 or 1 and 4. There is an increase in intersection delay when combining areas 1 and 3 by 6 percent.

Also, if the Basic model is applied to a combination of three areas -1 percent and -2 percent of reductions are made when combining areas 124 and 234, respectively.. No variation in intersection delay is obtained when combining areas 1, 2 and 3 and an increase of 1 percent is obtained when combining areas 1, 3 and 4.

The overall variation for the combination of areas 1, 2, 3 and 4 when applying the Basic Model is a reduction of -1 percent.

5.2.2 First Enhanced Model

When using the first enhanced model that applies HCM method for calculating pedestrian's delay, three important points are observed. The first one is that switching from the Basic Model, which gives vehicle delay a 100 percent relative weight, to the

Enhanced Model that gives 90 percent weight to vehicle delay at first, a general increase in intersection delay is observed as it can be seen in Figure 5.2 where the enhanced Model results and sensitivity analysis are compared alongside with the Basic Model.

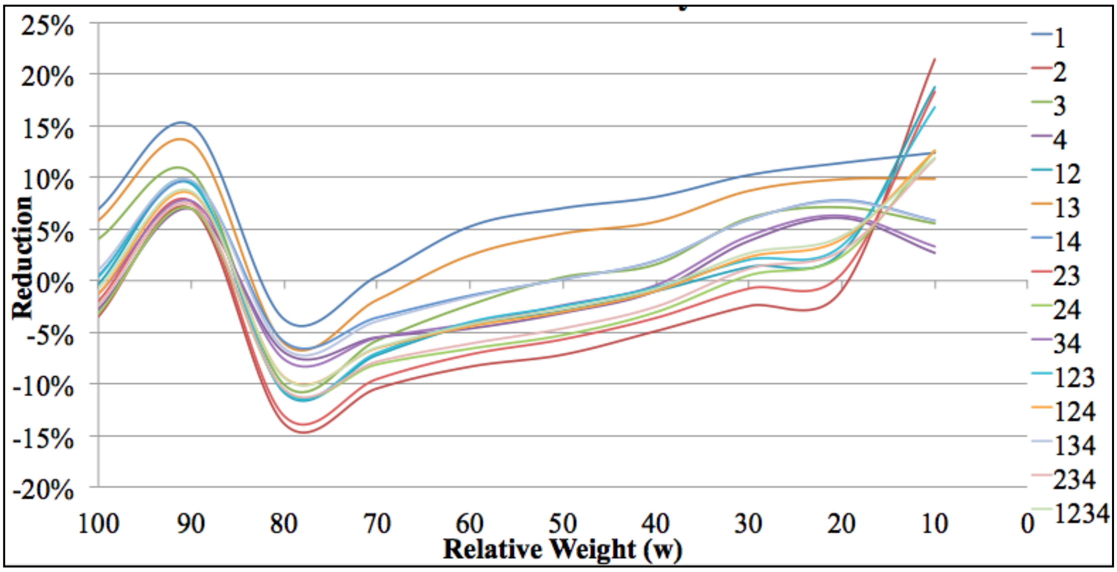

Figure 5.2. Variation of Reduction In Intersection Delay with Relative Weight in Each Group of Areas for the Basic and First Enhanced Models

A possible explanation is that the first reaction of the model when introducing pedestrian delay in the optimization is to increase intersection delay because another source of delay is involved into the equation and the results are affected by it. At that point of the sensitivity analysis the aggregation of another cause of delay makes intersection delay to go up.

The second important point is that from a 90 percent relative weight until some point close to 80 percent relative weight there is a generalized decrease in intersection delay. This means that the Enhanced Model is applied, considering the existence of pedestrians and passing a certain threshold where vehicle delay is still very powerful, intersection delay starts to go down. This shows the power of using a model that considers both pedestrian and vehicle delay, since it will be evaluating the interaction between both and that is something extremely important in the overall performance of the network. Again, the general pattern of having the greatest reductions in areas where vehicle presence is predominant is valid, since vehicle delay is still having a very high relative weight.

The third important thing is that there is a point when intersection delay starts to increase. This means that there is an optimal relative weight that achieves the maximum reductions and after that point results are getting worse. Also, as it can be seen in Figure 5.2, there is a pattern shift once vehicle delay is around 20 percent (which means that pedestrian delay is much more important, at 80 percent). The areas where vehicle presence is more important will now get higher increases in intersection delay than the areas where pedestrian presence is more important. This appears almost at the end of the sensitivity analysis, since until that point vehicle delay has been weighted much higher. Vehicle delay is obviously much more powerful in the model, equation 2-1, and a really high relative weight 1-*w* must be given to pedestrian delay in order to make the pattern shift.

The important result that is then obtained by analyzing the behavior of the first enhanced optimization model is that there is an optimal value of the relative weight that could be given to vehicle delay to produce the greatest reductions in intersection delay. Disagreeing with the intuitive conclusion that this value might depend on the volumes of vehicles and pedestrians that arrive to the intersection, there is a unique value that can be found independently of these volumes. This value is 78 percent and it is a common value for all of the studied areas. The reason why this value is unique is that the HCM model for intersection delay only captures vehicle volumes and not pedestrian volumes but at the same time can capture the interaction of both with the green time variable. Then an optimal value can be found that lets having the greatest reductions in intersection delay without looking at the vehicle and pedestrian volumes.

As commented, when applying the first Enhanced model varying the weight *w* between 90 percent and 20 percent the same order is maintained when looking at what areas or combination of areas have the highest reductions (or lowest increases) in intersection delay. When looking at results of individual areas it is obtained that between these two values of 90 percent and 20 percent for the weight, the best results are for areas 2, 3, 4 and 1 in that order. For example, at the system optimal (*w*=78 percent) the respective reductions for these areas are -14 percent, -10 percent, -7 percent and -4 percent, respectively. This order is maintained until *w*=20 percent where intersection delay is reduced in area 2 by -1 percent and increased in areas 3, 4 and 1 by 7 percent, 6 percent and 11 percent. As it can be seen in Figure 5.2, at this point the pattern changes and areas where pedestrian presence is higher (areas 3 and 1) start to get better results than areas where vehicle presence is more important. At *w*=10 percent the best results are obtained in area 4 with an increase of 3 percent while the worst result is obtained in area 2 (which had the best results until now) by an increase of 21 percent.

Focusing now on the results obtained in the optimal weight (*w*=78 percent) when looking at a combination of two areas reductions of intersection delay between -6 percent and -13 percent are obtained. Specifically, these reductions are -6 percent for a combination of areas 1 and 3, -6 percent for a combination of areas 1 and 4, -8 percent for a combination of areas 3 and 4, -11 percent for a combination of areas 2 and 4, -11 percent for a combination of areas 1 and 2 and -13 percent for a combination of areas 2 and 3.

When looking at a combination of three areas at the system optimal reductions between -7 percent and -11 percent are obtained. Combining areas 1, 3 and 4 a reduction of -7 percent, combining areas 1, 2 and 4 a reduction of -9 percent and the best results are obtained when combining areas 1, 2 and 3 or 2, 3 and 4 with reductions of -11 percent.

Applying the first enhanced model to the whole CBD, which means combining areas 1, 2, 3 and 4, a reduction in intersection delay of -9 percent is obtained

5.2.3 Second Enhanced Model

The second enhanced model, which recruits HSL method for computing pedestrian's delay, shows very different results than the first enhanced model. While the first enhanced Model has a strong pattern where optimal solutions can be found and where clear conclusions can be drawn, the second enhanced Model gives a steady and constant increase in delay as shown in Figure 5.3, together with the Basic Model results.

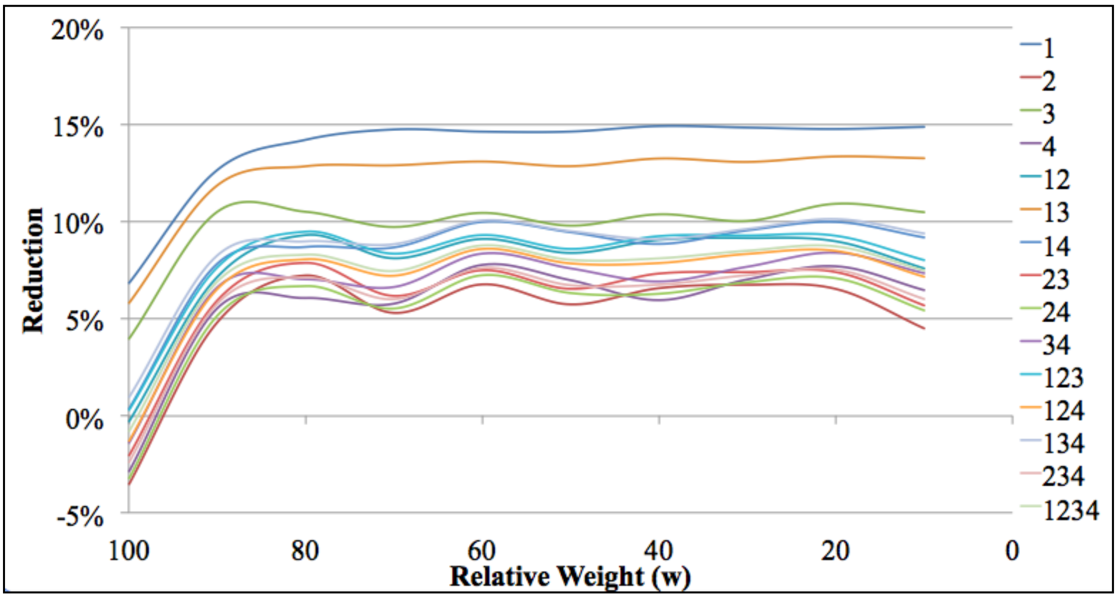

Figure 5.3. Variation of Reduction in Intersection Delay with Relative Weight in Each Group of Areas for the Basic and Second Enhanced Models

As it can be seen, the second enhanced optimization model increases intersection delay on all of the studied areas. The sensitivity analysis shows that although the objective delay function of the model is affected by the relative weight, there is something else that is making intersection delay reductions to be the same throughout the whole sensitivity analysis. The explanation of this result is in the initial objective function. The two ways pedestrian delay is calculated, as shown in Roshandeh *et al*. (2013) are the following:

The HCM method:

$$Delays_{PED,HCM} = \frac{\left[\left(T_G + T_Y + T_R\right) - g_{walk}\right]^2}{2 \cdot \left(T_G + T_Y + T_R\right)} \tag{5-1}$$

where

T_G, T_Y, T_R : Green, yellow and red times (sec); and

g_{walk} : Effective green time for pedestrians (sec).

The HSL method:

$$Delays_{PED,HSL} = \frac{\sum_{i=1}^{n}\left(P_{i2} \cdot \frac{T_{Ri}}{2}\right)}{\sum_{i=1}^{n}\left(P_{i1} + P_{i2}\right)} \qquad (5\text{-}2)$$

where

P_{i1} : Pedestrians crossing in phase *i* (ped);

P_{i2} : Pedestrians waiting in phase *i* (ped) ; and

T_{Ri} : Red time for interval *i* (sec).

As we can see, the HCM method takes into account the whole length of the cycle by adding green, red and yellow times, while the HSL method only takes red times. Since the optimization made by Roshandeh *et al*. (2013) maintains cycle length and intersection delay deals with cycle length too, when the HSL method is applied a performance is obtained that doesn't depend on the relative weight.

As commented, the HSL method gives constant increases in intersection delay throughout the whole sensitivity analysis. Take a look at this increase for each of the individual areas, it can be found that the greatest increase is in area 1 with around 15 percent, followed by area 3 with 10 percent, area 4 with 7 percent and finally area 2 with 6 percent.

When looking at a combination of two areas, increases between 6 percent and 13 percent are observed. The greatest increase is given in the combination of areas 1 and 3 with 13 percent increase, followed by combination of areas 1 and 4 with an increase of 9 percent, the combination of areas 1 and 2 with a 8 percent increase and finally combinations of areas 2 and 3, 2 and 4 or 3 and 4 with increases around 6 percent-7 percent.

For the combination of three areas results are very similar. The combination of areas 1, 3 and 4 gives the highest increase in intersection delay around 9 percent, followed by combination of areas 1, 2 and 3 with 8 percent and the combination of areas 1, 2 and 4 or 2, 3 and 4 with increase of around 7 percent.

When looking at how the second enhanced model on signal timing affects intersection delay to the combination of all areas 1, 2, 3 and 4 an increase of 8 percent is obtained.

5.3 Discussions

After analyzing the different results, the main quantitative overview is the following:

For the basic optimization model the greatest reductions are in areas 2 and 4 with reductions of -4 percent and -3 percent. The rest of the areas and the combination of them have reductions close to 0 percent.

For the first enhanced optimization model the highest values of reduction in intersection delay is obtained when a weight of 78 percent is given to vehicle delay in the objective function. Reductions between -5 percent and -15 percent are achieved, having the highest reduction on area 2. After that point, when giving more importance to pedestrian delay, intersection delay increases and at a certain point, around w=20 percent, the pattern suffers a shift and areas where pedestrian volume is higher tend to give better results than areas where vehicle volume is more important.

For the second enhanced optimization model a general increase of intersection delay is observed. Increases between 5 percent and 15 percent are achieved, having the highest values in area 1. The reductions do not depend on the value of w as it is shown in the sensitivity analysis.

CHAPTER 6

SUMMARY AND CONCLUSION

6.1 Summary

This thesis has studied the evolution of intersection delay in the city of Chicago when applying a traffic signal timing plan optimization model as proposed by Roshandeh *et al*. (2013). The model used for calculating intersection delay is based on the methodology proposed in the 2010 HCM, and it is applied to four different situations: the Original situation, the Basic Model Optimized situation, the first Enhanced Model Optimized situation (which used HCM to compute pedestrian's delay) and the second Enhanced Model Optimized situation (which used HSL to compute pedestrian's delay). The different optimized situations differ on whether or not they include pedestrian delay in the objective functions and the way in which pedestrian delay is calculated. The models includes the variable w, which is the relative weight of vehicle and pedestrian delay. A sensitivity analysis has been made in the calculation of intersection delay by varying the value of variable w and see if an optimal value of w could be found in order to obtain the greatest reductions in intersection delay.

After calculating delay for 934 intersections and analyzing results, it was observed that the first enhanced model can give reductions of as much as 15 percent in intersection delay. The value of the relative weight where this maximum reduction is obtained occurs for a value of w of 78 percent.

6.2 Conclusion

The main goal of this research was to study intersection delay after traffic signal timing plan optimization and to determine the optimal relative weight that could be given to vehicle delay in order to achieve the highest reductions. It can be concluded that by optimizing traffic signal timing plans, intersection delay is a good driver to test the efficiency of the model, since vehicle delay and travel time might not be useful for finding optimal situations. After analyzing the evolution of intersection delay, it is seen that it can go up or down depending on the relative weights of vehicle to pedestrian delays. Furthermore, it is observed that a weight of 78 percent should be given to vehicle delay when optimizing traffic signal timing plans in order to minimize intersection delay. This value maximizes the reductions between the original and the optimized situations.

The fact that this value is unique and doesn't depend on traffic or pedestrian volumes makes it a very powerful finding. This means that the analysis and results in this research could be used in any city in the world or at least in cities similar to Chicago. The obtained result can be used by city planners when designing traffic signal timing plans without having to use time and financial resources on traffic and pedestrian counting. Although results might not be the same in all areas of the city, the optimal value of w=78 percent will give the best results possible and should be used.

Further research could study and develop a model to simultaneously reduce vehicle, pedestrian and intersection delay. Intersection delay is able to capture the interaction between vehicles and pedestrians and is a good proxy to see the effectiveness of the optimization. Also, the use of other models for calculating intersection delay, especially in cities where the HCM hypotheses might not be true, could be interesting. The comparison of the results with the results in this research work could led to more solid conclusions on the exact weight that could be given to vehicle and pedestrian delay when optimizing traffic signal timing plans.

REFERENCES

Aashtiani H.Z., H. Iravani (1999). Use of intersection delay functions to improve reliability of traffic assignment models. *Proceedings of the 14th Annual International EMME/2 Conference, Chicago, Illinois*. 20-28.

Fambro D.B., N.M. Rouphail (2007). Generalized delay model for signalized intersections and arterial streets. *Transportation Research Record: Journal of the Transportation Research Board,* 1572, 112-121.

INRO (1994). *EMME/2 User's Manual, Software Release 7.* INRO Consultant, Montreal, Canada.

Koutsopoulos H.N., M. Habbal (1994). Effect of intersection delay modeling on the performance of traffic equilibrium models. *Transportation Research Part A*, 28(2), 133-149.

Li, Z., N. Li, F. Liu (2009). An effective calculating method of signalized intersection delay. *International Conference on Digital Image Processing 2009.* 211-215.

Liu, G.P., R.P. Zhai, Y.L. Pei (2007). A calculating method of intersection delay under signal control. *Intelligent Transportation Systems Conference, 2007. ISTC 2007, IEEE,* 648-653.

Lowrie, P.R. (1982). The Sydney co-ordinated adaptive traffic system- principles, methodology, algorithms. *Proceeding of the International Conference on Road Traffic Signaling,* 67-80.

Mazloumi, E., S. Moridpour, H. Mohsenian (2010). Delay function for signalized intersections in traffic assignment models. *Journal of Urban Planning and Development,* 136(1), 67-74.

Qiao F., P. Yi, H. Yang, S. Devarakonda (2002). Fuzzy logic based intersection delay estimation. *Mathematical and Computer Modelling,* 36(11-13), 1425-1434.

Quiroga C.A., D. Bullock (1999) Measuring control delay at signalized intersections. *Journal of Transportation Engineering,* 125(4), 271-280.

Roshandeh, A.M., H.S. Levinson, Z. Li, B. Zhou (2013). A new methodology for intersection signal timing optimization to simultaneously minimize vehicle and pedestrian delays. *ASCE Journal of Transportation Engineering.* (In Press).

TRB. (2010). 2010 *Highway Capacity Manual*. Transportation Research Board, National Academic Press Washington, D.C.

Webster F.V. (1958). *Traffic Signal Settings*. Road Research Laboratory, Crowthorne. United Kingdom.

Wolshon, B., W.C. Taylor (1999). Analysis of intersection delay under real-time adaptive signal control. *Transportation Research Part C*, 7(1), 53-72.

Zhu, Z., W. Liu, L. Liu, M. Cui, J. Li (2011). A simplified real-time road network model considering intersection delay and its application on vehicle navigation. *Applied Mechanics and Materials,* 58-60, 1959-1965.

APPENDIX A

REDUCTIONS ON INTERSECTION VEHICLE DELAY FOR EACH AREA USING THE BASIC MODEL

APPENDIX A: Reduction in Intersection Delay for Areas 1, 2, 3 and 4 Using the Basic Model

	AREA			
HOUR	**1**	**2**	**3**	**4**
00AM-1AM	-17%	-14%	-20%	-16%
01AM-2AM	-15%	-12%	-17%	-14%
02AM-03AM	-15%	-12%	-17%	-14%
03AM-04AM	-15%	-12%	-16%	-15%
04AM-05AM	-10%	-9%	-13%	-13%
05AM-06AM	2%	1%	-3%	-7%
06AM-07AM	14%	6%	9%	-1%
07AM-08AM	21%	10%	9%	3%
08AM-09AM	-1%	-16%	5%	-14%
09AM-10AM	24%	9%	27%	12%
10AM-11AM	28%	1%	18%	6%
11AM-12PM	25%	-5%	51%	-1%
12PM-01PM	24%	-1%	17%	1%
01PM-02PM	22%	-6%	9%	4%
02PM-03PM	19%	-35%	15%	0%
03PM-04PM	16%	13%	20%	19%
04PM-05PM	19%	-1%	11%	12%
05PM-06PM	15%	3%	9%	-3%
06PM-07PM	24%	7%	8%	3%
07PM-08PM	22%	20%	21%	10%
08PM-09PM	2%	1%	-2%	-2%
09PM-10PM	-10%	-8%	-11%	-11%
10PM-11PM	-15%	-12%	-16%	-14%
11PM-12PM	-16%	-12%	-18%	-14%

APPENDIX B

REDUCTIONS ON INTERSECTION VEHICLE AND PEDESTRIAN DELAY FOR EACH AREA USING THE FIRST ENHANCED MODEL WITH PEDESTRIAN DELAY CALCULATED USING THE HCM METHOD

APPENDIX B1: Reduction in Intersection Delay for Areas 1, 2, 3 and 4 Using a 10 Percent Weight for Vehicle Delay in the First Enhanced Model

	AREA			
HOUR	1	2	3	4
00AM-1AM	-21%	-16%	-23%	-17%
01AM-2AM	-19%	-14%	-20%	-16%
02AM-03AM	-18%	-15%	-20%	-16%
03AM-04AM	-18%	-14%	-19%	-16%
04AM-05AM	-14%	-12%	-16%	-15%
05AM-06AM	-4%	-5%	-8%	-10%
06AM-07AM	20%	9%	13%	2%
07AM-08AM	25%	9%	9%	7%
08AM-09AM	27%	498%	21%	9%
09AM-10AM	27%	-6%	16%	14%
10AM-11AM	44%	16%	15%	27%
11AM-12PM	44%	10%	28%	15%
12PM-01PM	36%	12%	14%	19%
01PM-02PM	40%	14%	39%	24%
02PM-03PM	32%	-20%	27%	8%
03PM-04PM	35%	36%	26%	30%
04PM-05PM	26%	12%	21%	18%
05PM-06PM	24%	15%	25%	10%
06PM-07PM	37%	12%	25%	17%
07PM-08PM	19%	8%	13%	4%
08PM-09PM	0%	-1%	-4%	-6%
09PM-10PM	-11%	-9%	-12%	-12%
10PM-11PM	-16%	-13%	-18%	-15%
11PM-12PM	-16%	-13%	-19%	-15%

APPENDIX B2: Reduction in Intersection Delay for Areas 1, 2, 3 and 4 Using a 20 Percent Weight for Vehicle Delay in the First Enhanced Model

	AREA			
HOUR	**1**	**2**	**3**	**4**
00AM-1AM	-22%	-18%	-23%	-18%
01AM-2AM	-19%	-16%	-20%	-16%
02AM-03AM	-19%	-16%	-20%	-16%
03AM-04AM	-19%	-15%	-19%	-17%
04AM-05AM	-15%	-13%	-16%	-15%
05AM-06AM	-5%	-7%	-9%	-11%
06AM-07AM	20%	9%	13%	1%
07AM-08AM	26%	7%	8%	7%
08AM-09AM	29%	-18%	18%	9%
09AM-10AM	27%	-3%	21%	14%
10AM-11AM	44%	18%	8%	15%
11AM-12PM	41%	12%	22%	22%
12PM-01PM	35%	13%	12%	15%
01PM-02PM	38%	16%	41%	25%
02PM-03PM	32%	-21%	20%	9%
03PM-04PM	36%	31%	29%	43%
04PM-05PM	26%	11%	92%	105%
05PM-06PM	24%	14%	22%	10%
06PM-07PM	37%	12%	21%	14%
07PM-08PM	13%	3%	9%	1%
08PM-09PM	-3%	-2%	-6%	-7%
09PM-10PM	-13%	-11%	-14%	-13%
10PM-11PM	-18%	-14%	-18%	-16%
11PM-12PM	-19%	-14%	-20%	-16%

APPENDIX B3: Reduction in Intersection Delay for Areas 1, 2, 3 and 4 Using a 30 Percent Weight for Vehicle Delay in the First Enhanced Model

	AREA			
HOUR	1	2	3	4
00AM-1AM	-22%	-18%	-23%	-18%
01AM-2AM	-20%	-16%	-20%	-17%
02AM-03AM	-19%	-17%	-21%	-17%
03AM-04AM	-19%	-16%	-20%	-17%
04AM-05AM	-16%	-14%	-17%	-15%
05AM-06AM	-6%	-8%	-10%	-12%
06AM-07AM	19%	7%	12%	0%
07AM-08AM	21%	6%	5%	3%
08AM-09AM	26%	-4%	21%	6%
09AM-10AM	24%	-3%	21%	7%
10AM-11AM	45%	15%	12%	13%
11AM-12PM	44%	13%	26%	15%
12PM-01PM	37%	16%	24%	16%
01PM-02PM	37%	14%	36%	22%
02PM-03PM	33%	-17%	25%	8%
03PM-04PM	29%	20%	22%	36%
04PM-05PM	22%	-2%	75%	93%
05PM-06PM	22%	4%	14%	5%
06PM-07PM	32%	5%	19%	15%
07PM-08PM	12%	3%	8%	1%
08PM-09PM	-4%	-4%	-7%	-6%
09PM-10PM	-14%	-11%	-14%	-13%
10PM-11PM	-19%	-15%	-19%	-16%
11PM-12PM	-19%	-15%	-21%	-16%

APPENDIX B4: Reduction in Intersection Delay for Areas 1, 2, 3 and 4 Using a 40 Percent Weight for Vehicle Delay in the First Enhanced Model

	AREA			
HOUR	1	2	3	4
00AM-1AM	-22%	-18%	-23%	-18%
01AM-2AM	-20%	-16%	-21%	-16%
02AM-03AM	-19%	-17%	-21%	-17%
03AM-04AM	-20%	-16%	-20%	-17%
04AM-05AM	-16%	-14%	-17%	-15%
05AM-06AM	-7%	-8%	-11%	-12%
06AM-07AM	16%	6%	10%	0%
07AM-08AM	19%	4%	-1%	2%
08AM-09AM	22%	-6%	19%	5%
09AM-10AM	17%	-6%	7%	4%
10AM-11AM	43%	7%	14%	22%
11AM-12PM	38%	6%	27%	20%
12PM-01PM	33%	11%	16%	15%
01PM-02PM	33%	8%	28%	18%
02PM-03PM	28%	-28%	24%	7%
03PM-04PM	29%	15%	27%	17%
04PM-05PM	20%	-2%	15%	10%
05PM-06PM	20%	2%	9%	-2%
06PM-07PM	27%	4%	15%	9%
07PM-08PM	10%	0%	5%	-2%
08PM-09PM	-5%	-5%	-8%	-7%
09PM-10PM	-14%	-12%	-15%	-13%
10PM-11PM	-19%	-16%	-20%	-16%
11PM-12PM	-19%	-15%	-21%	-16%

APPENDIX B5: Reduction in Intersection Delay for Areas 1, 2, 3 and 4 Using a 50 Percent Weight for Vehicle Delay in the First Enhanced Model

HOUR	1	2	3	4
00AM-1AM	-22%	-19%	-24%	-18%
01AM-2AM	-20%	-17%	-21%	-17%
02AM-03AM	-20%	-17%	-22%	-17%
03AM-04AM	-20%	-17%	-20%	-17%
04AM-05AM	-16%	-15%	-18%	-16%
05AM-06AM	-7%	-8%	-12%	-12%
06AM-07AM	16%	6%	9%	-2%
07AM-08AM	17%	3%	-1%	1%
08AM-09AM	18%	-4%	6%	2%
09AM-10AM	16%	-6%	17%	4%
10AM-11AM	39%	1%	13%	10%
11AM-12PM	38%	-2%	22%	12%
12PM-01PM	30%	-1%	15%	7%
01PM-02PM	31%	1%	17%	9%
02PM-03PM	27%	-30%	30%	7%
03PM-04PM	26%	4%	23%	13%
04PM-05PM	21%	-3%	15%	12%
05PM-06PM	18%	1%	6%	-2%
06PM-07PM	27%	2%	13%	8%
07PM-08PM	9%	0%	3%	-4%
08PM-09PM	-5%	-6%	-8%	-8%
09PM-10PM	-15%	-13%	-15%	-14%
10PM-11PM	-19%	-16%	-20%	-17%
11PM-12PM	-19%	-16%	-21%	-16%

APPENDIX B6: Reduction in Intersection Delay for Areas 1, 2, 3 and 4 Using a 60 Percent Weight for Vehicle Delay in the First Enhanced Model

	AREA			
HOUR	1	2	3	4
00AM-1AM	-22%	-19%	-24%	-19%
01AM-2AM	-20%	-17%	-21%	-17%
02AM-03AM	-20%	-17%	-22%	-17%
03AM-04AM	-20%	-17%	-20%	-17%
04AM-05AM	-16%	-15%	-18%	-16%
05AM-06AM	-7%	-9%	-12%	-12%
06AM-07AM	14%	4%	8%	-2%
07AM-08AM	17%	3%	-1%	0%
08AM-09AM	16%	-5%	7%	1%
09AM-10AM	13%	-9%	5%	2%
10AM-11AM	28%	-3%	2%	6%
11AM-12PM	34%	-5%	18%	7%
12PM-01PM	26%	-3%	12%	2%
01PM-02PM	26%	3%	14%	8%
02PM-03PM	23%	-34%	21%	4%
03PM-04PM	26%	3%	21%	12%
04PM-05PM	19%	-5%	10%	9%
05PM-06PM	16%	-3%	2%	-7%
06PM-07PM	26%	1%	12%	7%
07PM-08PM	8%	-2%	0%	-5%
08PM-09PM	-6%	-7%	-11%	-9%
09PM-10PM	-15%	-13%	-15%	-14%
10PM-11PM	-19%	-16%	-20%	-17%
11PM-12PM	-20%	-16%	-22%	-17%

APPENDIX B7: Reduction in Intersection Delay for Areas 1, 2, 3 and 4 Using a 70 Percent Weight for Vehicle Delay in the First Enhanced Model

	AREA			
HOUR	**1**	**2**	**3**	**4**
00AM-1AM	-23%	-19%	-24%	-19%
01AM-2AM	-21%	-17%	-21%	-17%
02AM-03AM	-21%	-18%	-22%	-17%
03AM-04AM	-21%	-17%	-21%	-17%
04AM-05AM	-17%	-15%	-18%	-16%
05AM-06AM	-8%	-9%	-12%	-12%
06AM-07AM	11%	1%	4%	-3%
07AM-08AM	6%	-2%	-9%	-2%
08AM-09AM	9%	-7%	-8%	-1%
09AM-10AM	3%	-14%	-2%	5%
10AM-11AM	16%	-6%	3%	6%
11AM-12PM	27%	-8%	10%	5%
12PM-01PM	19%	-6%	14%	1%
01PM-02PM	19%	-2%	4%	5%
02PM-03PM	16%	-38%	21%	3%
03PM-04PM	17%	-1%	14%	8%
04PM-05PM	8%	-9%	2%	6%
05PM-06PM	6%	-7%	-8%	-8%
06PM-07PM	19%	0%	2%	4%
07PM-08PM	7%	-2%	1%	-5%
08PM-09PM	-6%	-6%	-11%	-9%
09PM-10PM	-16%	-13%	-16%	-14%
10PM-11PM	-20%	-17%	-20%	-17%
11PM-12PM	-21%	-17%	-22%	-17%

APPENDIX B8: Reduction in Intersection Delay for Areas 1, 2, 3 and 4 Using a 80 Percent Weight for Vehicle Delay in the First Enhanced Model

	AREA			
HOUR	1	2	3	4
00AM-1AM	-23%	-19%	-24%	-19%
01AM-2AM	-21%	-18%	-21%	-17%
02AM-03AM	-21%	-18%	-22%	-17%
03AM-04AM	-21%	-18%	-21%	-18%
04AM-05AM	-17%	-15%	-18%	-16%
05AM-06AM	-9%	-10%	-13%	-12%
06AM-07AM	8%	-2%	-3%	-6%
07AM-08AM	4%	-6%	-11%	-4%
08AM-09AM	2%	-12%	-9%	-1%
09AM-10AM	-6%	-17%	-13%	-1%
10AM-11AM	13%	-9%	-9%	4%
11AM-12PM	18%	-17%	6%	8%
12PM-01PM	12%	-16%	0%	-3%
01PM-02PM	13%	-12%	-8%	0%
02PM-03PM	7%	-49%	-1%	-2%
03PM-04PM	6%	-6%	3%	5%
04PM-05PM	-2%	-15%	17%	6%
05PM-06PM	-4%	-13%	-15%	-10%
06PM-07PM	7%	-6%	-8%	-1%
07PM-08PM	7%	-3%	0%	-5%
08PM-09PM	-6%	-6%	-12%	-9%
09PM-10PM	-16%	-14%	-16%	-14%
10PM-11PM	-20%	-17%	-20%	-17%
11PM-12PM	-21%	-17%	-22%	-17%

APPENDIX B9: Reduction in Intersection Delay for Areas 1, 2, 3 and 4 Using a 90 Percent Weight for Vehicle Delay in the First Enhanced Model

	AREA			
HOUR	1	2	3	4
00AM-1AM	-18%	-14%	-20%	-16%
01AM-2AM	-15%	-11%	-17%	-14%
02AM-03AM	-15%	-11%	-17%	-14%
03AM-04AM	-15%	-11%	-16%	-14%
04AM-05AM	-11%	-8%	-12%	-12%
05AM-06AM	2%	1%	-3%	-7%
06AM-07AM	18%	11%	14%	2%
07AM-08AM	29%	19%	23%	11%
08AM-09AM	39%	31%	39%	20%
09AM-10AM	36%	21%	30%	21%
10AM-11AM	38%	23%	26%	25%
11AM-12PM	36%	17%	28%	24%
12PM-01PM	36%	19%	21%	19%
01PM-02PM	37%	20%	24%	27%
02PM-03PM	39%	-25%	35%	20%
03PM-04PM	36%	31%	32%	34%
04PM-05PM	37%	22%	39%	45%
05PM-06PM	35%	26%	29%	10%
06PM-07PM	32%	21%	24%	17%
07PM-08PM	20%	13%	19%	9%
08PM-09PM	4%	2%	-1%	-3%
09PM-10PM	-10%	-7%	-11%	-10%
10PM-11PM	-14%	-11%	-16%	-14%
11PM-12PM	-16%	-11%	-18%	-14%

APPENDIX C

REDUCTIONS IN INTERSECTION VEHICLE AND PEDESTRIAN DELAY FOR EACH AREA USING THE SECOND ENHANCED MODEL WITH PEDESTRIAN DELAY ESTIMATED USING THE HSL METHOD

APPENDIX C1: Reduction in Intersection Delay for Areas 1, 2, 3 and 4 Using a 10 Percent Weight for Vehicle Delay in the Second Enhanced Model

	AREA			
HOUR	1	2	3	4
00AM-1AM	-18%	-14%	-20%	-16%
01AM-2AM	-15%	-11%	-17%	-14%
02AM-03AM	-15%	-11%	-17%	-14%
03AM-04AM	-15%	-11%	-16%	-14%
04AM-05AM	-10%	-8%	-12%	-12%
05AM-06AM	2%	1%	-3%	-7%
06AM-07AM	18%	11%	13%	1%
07AM-08AM	29%	19%	25%	11%
08AM-09AM	39%	-19%	37%	24%
09AM-10AM	37%	25%	39%	28%
10AM-11AM	37%	20%	25%	26%
11AM-12PM	35%	18%	34%	33%
12PM-01PM	35%	20%	24%	20%
01PM-02PM	35%	18%	22%	22%
02PM-03PM	38%	-31%	32%	20%
03PM-04PM	38%	26%	31%	27%
04PM-05PM	36%	24%	30%	25%
05PM-06PM	35%	25%	30%	11%
06PM-07PM	32%	21%	24%	15%
07PM-08PM	21%	13%	19%	10%
08PM-09PM	3%	2%	-1%	-3%
09PM-10PM	-10%	-7%	-10%	-10%
10PM-11PM	-14%	-11%	-16%	-13%
11PM-12PM	-16%	-11%	-18%	-14%

APPENDIX C2: Reduction in Intersection Delay for Areas 1, 2, 3 and 4 Using a 20 Percent Weight for Vehicle Delay in the Second Enhanced Model

	AREA			
HOUR	1	2	3	4
00AM-1AM	-18%	-14%	-20%	-16%
01AM-2AM	-15%	-11%	-17%	-14%
02AM-03AM	-15%	-11%	-17%	-14%
03AM-04AM	-15%	-11%	-16%	-14%
04AM-05AM	-11%	-8%	-12%	-12%
05AM-06AM	2%	1%	-3%	-7%
06AM-07AM	18%	11%	13%	2%
07AM-08AM	28%	19%	23%	11%
08AM-09AM	38%	28%	34%	18%
09AM-10AM	34%	17%	25%	20%
10AM-11AM	37%	23%	23%	24%
11AM-12PM	36%	20%	28%	28%
12PM-01PM	35%	20%	28%	19%
01PM-02PM	38%	24%	26%	29%
02PM-03PM	36%	-36%	30%	21%
03PM-04PM	36%	29%	33%	30%
04PM-05PM	37%	24%	59%	67%
05PM-06PM	34%	25%	27%	9%
06PM-07PM	33%	21%	25%	16%
07PM-08PM	21%	13%	19%	9%
08PM-09PM	3%	2%	-1%	-2%
09PM-10PM	-10%	-7%	-11%	-10%
10PM-11PM	-14%	-11%	-16%	-13%
11PM-12PM	-16%	-11%	-18%	-14%

APPENDIX C3: Reduction in Intersection Delay for Areas 1, 2, 3 and 4 Using a 30 Percent Weight for Vehicle Delay in the Second Enhanced Model

	AREA			
HOUR	1	2	3	4
00AM-1AM	-18%	-14%	-20%	-16%
01AM-2AM	-15%	-11%	-17%	-14%
02AM-03AM	-15%	-11%	-17%	-14%
03AM-04AM	-15%	-11%	-16%	-14%
04AM-05AM	-11%	-8%	-12%	-12%
05AM-06AM	2%	1%	-3%	-7%
06AM-07AM	18%	11%	13%	2%
07AM-08AM	29%	19%	23%	10%
08AM-09AM	37%	27%	30%	18%
09AM-10AM	35%	20%	25%	21%
10AM-11AM	36%	22%	23%	26%
11AM-12PM	37%	19%	28%	21%
12PM-01PM	34%	21%	27%	23%
01PM-02PM	38%	21%	30%	26%
02PM-03PM	39%	-31%	35%	20%
03PM-04PM	38%	32%	33%	35%
04PM-05PM	35%	24%	31%	50%
05PM-06PM	35%	25%	26%	9%
06PM-07PM	33%	21%	27%	17%
07PM-08PM	21%	13%	19%	9%
08PM-09PM	3%	2%	-1%	-3%
09PM-10PM	-9%	-6%	-10%	-10%
10PM-11PM	-14%	-11%	-16%	-13%
11PM-12PM	-16%	-11%	-18%	-14%

APPENDIX C4: Reduction in Intersection Delay for Areas 1, 2, 3 and 4 Using a 40 Percent Weight for Vehicle Delay in the Second Enhanced Model

	AREA			
HOUR	1	2	3	4
00AM-1AM	-18%	-14%	-20%	-16%
01AM-2AM	-16%	-11%	-17%	-14%
02AM-03AM	-15%	-11%	-17%	-14%
03AM-04AM	-15%	-11%	-16%	-14%
04AM-05AM	-11%	-8%	-12%	-12%
05AM-06AM	2%	1%	-3%	-7%
06AM-07AM	18%	11%	13%	1%
07AM-08AM	28%	18%	23%	11%
08AM-09AM	40%	26%	38%	23%
09AM-10AM	38%	22%	44%	28%
10AM-11AM	38%	23%	26%	24%
11AM-12PM	35%	19%	26%	28%
12PM-01PM	35%	20%	24%	20%
01PM-02PM	39%	23%	24%	18%
02PM-03PM	37%	-31%	35%	21%
03PM-04PM	36%	26%	31%	26%
04PM-05PM	36%	22%	28%	25%
05PM-06PM	34%	26%	25%	9%
06PM-07PM	33%	20%	25%	15%
07PM-08PM	21%	14%	19%	10%
08PM-09PM	3%	2%	-1%	-3%
09PM-10PM	-10%	-7%	-10%	-10%
10PM-11PM	-14%	-11%	-16%	-14%
11PM-12PM	-16%	-11%	-18%	-14%

APPENDIX C5: Reduction in Intersection Delay for Areas 1, 2, 3 and 4 Using a 50 Percent Weight for Vehicle Delay in the Second Enhanced Model

	AREA			
HOUR	1	2	3	4
00AM-1AM	-18%	-14%	-20%	-16%
01AM-2AM	-16%	-11%	-17%	-14%
02AM-03AM	-15%	-11%	-17%	-14%
03AM-04AM	-15%	-11%	-16%	-14%
04AM-05AM	-11%	-8%	-12%	-12%
05AM-06AM	2%	1%	-3%	-7%
06AM-07AM	18%	11%	14%	2%
07AM-08AM	29%	19%	24%	11%
08AM-09AM	38%	30%	32%	18%
09AM-10AM	38%	24%	36%	30%
10AM-11AM	37%	22%	29%	28%
11AM-12PM	35%	18%	25%	26%
12PM-01PM	35%	21%	28%	20%
01PM-02PM	38%	17%	22%	19%
02PM-03PM	36%	-51%	24%	18%
03PM-04PM	35%	27%	28%	28%
04PM-05PM	36%	22%	33%	48%
05PM-06PM	33%	24%	27%	11%
06PM-07PM	34%	21%	24%	15%
07PM-08PM	20%	13%	19%	10%
08PM-09PM	3%	2%	0%	-2%
09PM-10PM	-10%	-7%	-11%	-10%
10PM-11PM	-14%	-11%	-16%	-13%
11PM-12PM	-16%	-11%	-18%	-14%

APPENDIX C6: Reduction in Intersection Delay for Areas 1, 2, 3 and 4 Using a 60 Percent Weight for Vehicle Delay in the Second Enhanced Model

	AREA			
HOUR	1	2	3	4
00AM-1AM	-19%	-14%	-20%	-16%
01AM-2AM	-16%	-11%	-17%	-14%
02AM-03AM	-16%	-11%	-17%	-14%
03AM-04AM	-16%	-11%	-16%	-14%
04AM-05AM	-12%	-8%	-12%	-12%
05AM-06AM	0%	1%	-3%	-7%
06AM-07AM	18%	11%	14%	2%
07AM-08AM	29%	19%	24%	11%
08AM-09AM	40%	24%	39%	22%
09AM-10AM	36%	22%	29%	25%
10AM-11AM	37%	23%	24%	28%
11AM-12PM	36%	20%	33%	27%
12PM-01PM	36%	21%	24%	22%
01PM-02PM	39%	22%	24%	25%
02PM-03PM	36%	-29%	28%	18%
03PM-04PM	37%	29%	37%	31%
04PM-05PM	38%	24%	40%	56%
05PM-06PM	33%	24%	25%	11%
06PM-07PM	33%	21%	24%	16%
07PM-08PM	20%	13%	18%	9%
08PM-09PM	3%	2%	0%	-2%
09PM-10PM	-10%	-7%	-11%	-10%
10PM-11PM	-15%	-11%	-16%	-14%
11PM-12PM	-16%	-11%	-18%	-14%

APPENDIX C7: Reduction in Intersection Delay for Areas 1, 2, 3 and 4 Using a 70 Percent Weight for Vehicle Delay in the Second Enhanced Model

	AREA			
HOUR	1	2	3	4
00AM-1AM	-19%	-14%	-20%	-16%
01AM-2AM	-16%	-11%	-17%	-14%
02AM-03AM	-16%	-11%	-17%	-14%
03AM-04AM	-16%	-11%	-16%	-14%
04AM-05AM	-12%	-8%	-13%	-12%
05AM-06AM	0%	1%	-3%	-7%
06AM-07AM	18%	11%	13%	1%
07AM-08AM	30%	19%	24%	11%
08AM-09AM	40%	34%	35%	15%
09AM-10AM	35%	21%	26%	19%
10AM-11AM	41%	23%	23%	29%
11AM-12PM	35%	18%	33%	27%
12PM-01PM	36%	20%	25%	20%
01PM-02PM	38%	24%	25%	27%
02PM-03PM	38%	-70%	35%	20%
03PM-04PM	36%	27%	27%	30%
04PM-05PM	37%	22%	27%	25%
05PM-06PM	35%	26%	29%	8%
06PM-07PM	32%	21%	25%	16%
07PM-08PM	20%	13%	19%	9%
08PM-09PM	2%	2%	-1%	-3%
09PM-10PM	-10%	-7%	-10%	-10%
10PM-11PM	-15%	-11%	-16%	-13%
11PM-12PM	-16%	-11%	-18%	-14%

APPENDIX C8: Reduction in Intersection Delay for Areas 1, 2, 3 and 4 Using a 80 Percent Weight for Vehicle Delay in the Second Enhanced Model

	AREA			
HOUR	**1**	**2**	**3**	**4**
00AM-1AM	-20%	-14%	-20%	-16%
01AM-2AM	-17%	-11%	-17%	-14%
02AM-03AM	-16%	-11%	-17%	-14%
03AM-04AM	-16%	-11%	-16%	-14%
04AM-05AM	-12%	-8%	-13%	-12%
05AM-06AM	0%	1%	-3%	-7%
06AM-07AM	18%	11%	14%	2%
07AM-08AM	29%	19%	24%	11%
08AM-09AM	41%	31%	33%	18%
09AM-10AM	35%	19%	28%	24%
10AM-11AM	39%	23%	29%	30%
11AM-12PM	36%	21%	29%	28%
12PM-01PM	36%	19%	25%	17%
01PM-02PM	37%	25%	23%	23%
02PM-03PM	37%	-26%	31%	18%
03PM-04PM	38%	33%	46%	30%
04PM-05PM	35%	22%	28%	28%
05PM-06PM	34%	25%	34%	9%
06PM-07PM	33%	20%	23%	17%
07PM-08PM	18%	13%	18%	9%
08PM-09PM	1%	2%	-1%	-2%
09PM-10PM	-11%	-7%	-11%	-10%
10PM-11PM	-16%	-11%	-16%	-14%
11PM-12PM	-17%	-11%	-18%	-14%

APPENDIX C9: Reduction in Intersection Delay for Areas 1, 2, 3 and 4 Using a 90 Percent Weight for Vehicle Delay in the Second Enhanced Model

	AREA			
HOUR	**1**	**2**	**3**	**4**
00AM-1AM	-20%	-14%	-20%	-16%
01AM-2AM	-17%	-11%	-17%	-14%
02AM-03AM	-17%	-11%	-17%	-14%
03AM-04AM	-16%	-11%	-16%	-14%
04AM-05AM	-12%	-8%	-13%	-12%
05AM-06AM	-1%	1%	-3%	-7%
06AM-07AM	17%	11%	13%	2%
07AM-08AM	28%	19%	24%	11%
08AM-09AM	37%	33%	36%	19%
09AM-10AM	33%	22%	35%	28%
10AM-11AM	32%	20%	23%	21%
11AM-12PM	31%	19%	32%	26%
12PM-01PM	33%	18%	27%	21%
01PM-02PM	34%	18%	22%	19%
02PM-03PM	31%	-67%	29%	19%
03PM-04PM	36%	23%	44%	25%
04PM-05PM	34%	24%	27%	24%
05PM-06PM	35%	25%	31%	10%
06PM-07PM	30%	20%	23%	16%
07PM-08PM	18%	13%	19%	9%
08PM-09PM	2%	2%	-1%	-2%
09PM-10PM	-11%	-7%	-11%	-10%
10PM-11PM	-16%	-11%	-16%	-14%
11PM-12PM	-17%	-11%	-18%	-14%

APPENDIX D

MATLAB CODE FOR CALCULATING INTERSECTION DELAY

```
%Program to calculate d1 and d2 %%%THIS IS ONLY FOR ONE HOUR!!
BNodeLink=xlsread('BeforeBasic_info','BNodeLink');
numnode=size(BNodeLink);
n=numnode(1);     %number of nodes
q=numnode(2);   %number of columns in the node-link column (not
the same as number of links since some are empty)
numlink=zeros(n,1);
CycleLength=xlsread('BeforeBasic_info','CycleLength'); %reads
CycleLength sheet and gets node+cyclelenght
CycleLength=CycleLength(:,2);   %Column vector for cycle lenghts
VolumeAB=xlsread('BeforeBasic_Vol','Vol2324'); %%reads VolumeAB
sheet and gets node+volume  %CHANGE
VolumeAB=VolumeAB(1:n,2:q);            %deletes node column
VolumeNode=(sum(VolumeAB'))';           %column vector for the
total in_link volume in each node
CapacityAB=xlsread('BeforeBasic_info','CapacityAB'); %%reads
CapacityAB sheet and gets node+volume
CapacityAB=CapacityAB(1:n,2:q);            %deletes node column
LaneThru=xlsread('BeforeBasic_info','LaneThru'); %%reads LaneThru
sheet and gets node+volume
LaneThru=LaneThru(1:n,2:q);            %deletes node column
LaneLeft=xlsread('BeforeBasic_info','LaneLeft'); %%reads LaneLeft
sheet and gets node+volume
LaneLeft=LaneLeft(1:n,2:q);            %deletes node column
LaneRight=xlsread('BeforeBasic_info','LaneRight'); %%reads
LaneRight sheet and gets node+volume
LaneRight=LaneRight(1:n,2:q);            %deletes node column
LaneTotal=LaneThru+LaneLeft+LaneRight;  %total number of lanes in
the approach
GreenThru=xlsread('BeforeBasic_info','GreenThru'); %%reads
GreenThru sheet and gets node+volume
GreenThru=GreenThru(1:n,2:q);            %deletes node column
GreenLeft=xlsread('BeforeBasic_info','GreenLeft'); %%reads
GreenLeft sheet and gets node+volume
GreenLeft=GreenLeft(1:n,2:q);            %deletes node column
GreenRight=xlsread('BeforeBasic_info','GreenRight'); %%reads
GreenRight sheet and gets node+volume
GreenRight=GreenRight(1:n,2:q);            %deletes node column
X=zeros(n,q-1);
d1thru=zeros(n,q-1);
d2thru=zeros(n,q-1);
d1left=zeros(n,q-1);
d1right=zeros(n,q-1);
d1app=zeros(n,q-1);
d1nodeprev=zeros(n,q-1);
d2nodeprev=zeros(n,q-1);
for i=1:n
    for j=2:q
        if BNodeLink(i,j)>0 %if the column contains a link, count
        numlink(i,1)=numlink(i,1)+1;
        else
        numlink(i,1)=numlink(i,1); %if it doesnt contain a link,
don't count
        end %numlink is a column vector containg the number of
links in each node
    end
    for k=1:numlink(i)
        X(i,k)=VolumeAB(i,k)./CapacityAB(i,k); %matrix containing
x for each link and each node
        T=1;    % time of observation 1 hour
```

```
        K=0.5;  % pretimed signals
        I=1;    % we consider each intersection as isolated
        %%%D1 and D2 CALCULATION
        if LaneThru(i,k)>0  %will calculate dthru if there are
thru lanes only
            d1thru(i,k)=(0.5.*CycleLength(i,1).*(1-
GreenThru(i,k)/CycleLength(i,1)).^2)/(1-
(min(1,X(i,k))).*(GreenThru(i,k)./CycleLength(i,1)));
            d2thru(i,k)=900.*T.*((X(i,k)-1)+((X(i,k)-
1).^2+(8.*K.*I.*X(i,k)./(CycleLength(i,1).*T)))^(1/2));
        else
            d1thru(i,k)=0;
        end
        if LaneLeft(i,k)>0  %will calculate dleft if there are
left lanes only
            d1left(i,k)=(0.5.*CycleLength(i,1).*(1-
GreenLeft(i,k)/CycleLength(i,1)).^2)/(1-
(min(1,X(i,k))).*(GreenLeft(i,k)./CycleLength(i,1)));
        else
            d1left(i,k)=0;
        end
        if LaneRight(i,k)>0  %will calculate dthru if there are
right lanes only
            d1right(i,k)=(0.5.*CycleLength(i,1).*(1-
GreenRight(i,k)/CycleLength(i,1)).^2)/(1-
(min(1,X(i,k))).*(GreenRight(i,k)./CycleLength(i,1)));
        else
            d1right(i,k)=0;
        end
        d2left=d2thru;
        d2right=d2thru;
        % We now calculate average delay at the approach
weighting by
        % number of lanes (because we suppose equal distribution
of traffic
        % per lane

d1app(i,k)=d1thru(i,k).*(LaneThru(i,k)./LaneTotal(i,k))+d1left(i,
k).*(LaneLeft(i,k)./LaneTotal(i,k))+d1right(i,k).*(LaneRight(i,k)
./LaneTotal(i,k));
        d2app=d2thru;
        %Now we calculate delay for the intersection weighting by
traffic
        %volume in each approach

d1nodeprev(i,k)=d1app(i,k).*VolumeAB(i,k)./VolumeNode(i,1);

d2nodeprev(i,k)=d2app(i,k).*VolumeAB(i,k)./VolumeNode(i,1);
    end
end
d1node=(sum(d1nodeprev'))';
d2node=(sum(d2nodeprev'))';
%%Program to calculate d3 %%%THIS IS ONLY FOR ONE HOUR!!
BNodeLink=xlsread('BeforeBasic_info','BNodeLink');
numnode=size(BNodeLink);
n=numnode(1);     %number of nodes
q=numnode(2);   %number of columns in the node-link column (not
the same as number of links since some are empty)
numlink=zeros(n,1);
```

```
CapacityAB=xlsread('BeforeBasic_info','CapacityAB'); %%reads
CapacityAB sheet and gets node+volume
CapacityAB=CapacityAB(1:n,2:q);            %deletes node column
VolumeAB1=xlsread('BeforeBasic_Vol','Vol0001');
VolumeAB1=VolumeAB1(1:n,2:q);            %deletes node column
VolumeAB2=xlsread('BeforeBasic_Vol','Vol0102');
VolumeAB2=VolumeAB2(1:n,2:q);            %deletes node column
VolumeAB3=xlsread('BeforeBasic_Vol','Vol0203');
VolumeAB3=VolumeAB3(1:n,2:q);            %deletes node column
VolumeAB4=xlsread('BeforeBasic_Vol','Vol0304');
VolumeAB4=VolumeAB4(1:n,2:q);            %deletes node column
VolumeAB5=xlsread('BeforeBasic_Vol','Vol0405');
VolumeAB5=VolumeAB5(1:n,2:q);            %deletes node column
VolumeAB6=xlsread('BeforeBasic_Vol','Vol0506');
VolumeAB6=VolumeAB6(1:n,2:q);            %deletes node column
VolumeAB7=xlsread('BeforeBasic_Vol','Vol0607');
VolumeAB7=VolumeAB7(1:n,2:q);            %deletes node column
VolumeAB8=xlsread('BeforeBasic_Vol','Vol0708');
VolumeAB8=VolumeAB8(1:n,2:q);            %deletes node column
VolumeAB9=xlsread('BeforeBasic_Vol','Vol0809');
VolumeAB9=VolumeAB9(1:n,2:q);            %deletes node column
VolumeAB10=xlsread('BeforeBasic_Vol','Vol0910');
VolumeAB10=VolumeAB10(1:n,2:q);            %deletes node column
VolumeAB11=xlsread('BeforeBasic_Vol','Vol1011');
VolumeAB11=VolumeAB11(1:n,2:q);            %deletes node column
VolumeAB12=xlsread('BeforeBasic_Vol','Vol1112');
VolumeAB12=VolumeAB12(1:n,2:q);            %deletes node column
VolumeAB13=xlsread('BeforeBasic_Vol','Vol1213');
VolumeAB13=VolumeAB13(1:n,2:q);            %deletes node column
VolumeAB14=xlsread('BeforeBasic_Vol','Vol1314');
VolumeAB14=VolumeAB14(1:n,2:q);            %deletes node column
VolumeAB15=xlsread('BeforeBasic_Vol','Vol1415');
VolumeAB15=VolumeAB15(1:n,2:q);            %deletes node column
VolumeAB16=xlsread('BeforeBasic_Vol','Vol1516');
VolumeAB16=VolumeAB16(1:n,2:q);            %deletes node column
VolumeAB17=xlsread('BeforeBasic_Vol','Vol1617');
VolumeAB17=VolumeAB17(1:n,2:q);            %deletes node column
VolumeAB18=xlsread('BeforeBasic_Vol','Vol1718');
VolumeAB18=VolumeAB18(1:n,2:q);            %deletes node column
VolumeAB19=xlsread('BeforeBasic_Vol','Vol1819');
VolumeAB19=VolumeAB19(1:n,2:q);            %deletes node column
VolumeAB20=xlsread('BeforeBasic_Vol','Vol1920');
VolumeAB20=VolumeAB20(1:n,2:q);            %deletes node column
VolumeAB21=xlsread('BeforeBasic_Vol','Vol2021');
VolumeAB21=VolumeAB21(1:n,2:q);            %deletes node column
VolumeAB22=xlsread('BeforeBasic_Vol','Vol2122');
VolumeAB22=VolumeAB22(1:n,2:q);            %deletes node column
VolumeAB23=xlsread('BeforeBasic_Vol','Vol2223');
VolumeAB23=VolumeAB23(1:n,2:q);            %deletes node column
VolumeAB24=xlsread('BeforeBasic_Vol','Vol2324');
VolumeAB24=VolumeAB24(1:n,2:q);            %deletes node column
Qb1=zeros(n,q-1);Qb2=zeros(n,q-1);
Qeo1=zeros(n,q-1);Qeo2=zeros(n,q-1);
ta1=zeros(n,q-1);ta2=zeros(n,q-1);
Qe1=zeros(n,q-1);Qe2=zeros(n,q-1);
T=1; %This is t and T and is one hour (duration of the analysis
and time where volume is constant)
t=ones(n,q-1);
d3nodeprev=zeros(n,q-1);
for i=1:n
```

```
    for j=2:q
        if BNodeLink(i,j)>0 %if the column contains a link, count
        numlink(i,1)=numlink(i,1)+1;
        else
        numlink(i,1)=numlink(i,1); %if it doesnt contain a link,
don't count
        end %numlink is a column vector containg the number of
links in each node
    end
    for k=1:numlink(i)
        Qb2(i,k)=max(Qb1(i,k)-(CapacityAB(i,k)-
VolumeAB24(i,k)).*T,0);       %CHANGE
        if
VolumeAB24(i,k)>CapacityAB(i,k)
 %CHANGE
            ta2(i,k)=t(i,k);
            Qeo2(i,k)=ta2(i,k).*(VolumeAB24(i,k)-
CapacityAB(i,k));           %CHANGE
        else
            ta2(i,k)=Qb2(i,k)./(CapacityAB(i,k)-
VolumeAB24(i,k));            %CHANGE
        end
        Qe2(i,k)=Qb2(i,k)+ta2(i,k).*(VolumeAB24(i,k)-
CapacityAB(i,k));       %CHANGE

d3nodeprev(i,k)=(3600/VolumeAB24(i,k).*T).*((ta2(i,k)./2).*(Qb2(i
,k)+Qe2(i,k)-Qeo2(i,k))+(Qe2(i,k).^2-
Qeo2(i,k).^2)./(2.*CapacityAB(i,k))-
(Qb2(i,k).^2)./(2.*CapacityAB(i,k)));
    end
end
d3node=(sum(d3nodeprev'))';
dnode=d1node+d2node+d3node
```